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U. S. DEPARTMENT OF AGRICULTURE.

DIVISION OF FORESTRY.

TIMBER PHYSICS SERIES.

ECONOMICAL DESIGNING OF TIMBER TRESTLE BRIDGES,

BY

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LETTER OF TRANSMITTAL.

U. S. DEPARTMENT OF AGRICULTURE,
DIVISION OF FORESTRY,
Washington, D. C., October 4, 1895.

SIR: I have the honor to transmit a paper on "Economical designing of timber trestle bridges," prepared by Mr. A. L. Johnson as a result of his studies of the values of strength developed in the timber-test work of this division. The paper includes also tables of safe loads for beams, columns, bearing areas, etc., applicable to the designing of timber structures in general.

As the author states, not only large savings, counting by millions of dollars, may be effected by the proposed change in the present practice, but great economy in the use of our most valuable timber species would result at the same time. The information thus conveyed will, therefore, it is hoped, tend toward a more rational use of our forest resources in general and will recommend itself to the railroad companies in particular.

That there may be no question as to the soundness of the deductions and the value of the paper to the professional reader, the same has been submitted to two leading bridge engineers, and their opinions are appended; also a report upon the strength of bridge and trestle timbers by a committee of the American International Association of Railway Superintendents of Bridges and Buildings. Those statements will, it is believed, greatly add to the value of the publication.

I recommend the publication of these papers in a single pamphlet, as Bulletin No. 12 of this division.

Respectfully,

B. E. FERNOW,
Chief of Division of Forestry

Hon. J. STERLING MORTON,
Secretary of Agriculture.

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ECONOMICAL DESIGNING OF TIMBER TRESTLE BRIDGES.

INTRODUCTION.

There are in the United States at least 2,000 miles of timber trestle, representing an expenditure of more than $60,000,000. These have to be entirely replaced every nine years, on the average, making an annual expenditure of about $7,000,000, which, capitalized at 4 per cent, gives an invested capital of $175,000,000 necessary to maintain these structures, consuming annually about 260,000,000 feet, B. M., of timber, nearly all of it being in large sizes, very valuable for other purposes.

Any economy effected, upon however small a margin it is based, is therefore, when multiplied so enormously, of great benefit not only to the railroad companies but to our national forestry interests.

The capital invested in timber structures is greatly in excess (probably twice as much) of that invested in structures of iron and steel. Every piece in these latter structures is thoroughly inspected, both chemically and physically, and carefully designed to carry the imposed load. Timber structures, on the other hand, have been designed according to the general principal that the Lord takes care of His own, as the great number of fatalities resulting from failures of these structures will attest. Experience is not only a dear teacher, but requires endless time to tell what he knows. That this time has not yet expired is evidenced by the anachronisms of design existing in the present practice.

It is a well-known principle that a chain is no stronger than its weakest link, and therefore, having one weak link, it is unnecessary that the others should be any stronger. Yet there are thousands of timber structures throughout this country to-day that have, in some parts, barely sufficient strength to carry their working load (with a factor of safety of only 1—see table on pages 9 and 10), while other portions of the same structure have from twenty to thirty times this strength. The engineer who would design an iron structure in such a manner would be the laughing stock of the most ignorant man in the profession.

There are now about 800 miles of timber trestle bridges in this country so designed. What is the cause of this almost universal disregard of the laws of economy? One reason is that timber has been in the past very cheap, and it was not necessary to consider sizes so carefully as is done in iron and steel structures. The principal reason, probably,

is that material even of the same species was found to give strength factors of such a wide range of values that scientific investigation was discouraged. The elements affecting the strength were not known, and there was no assurance that the piece to be used would develop a strength within narrow limits comparable with that shown by the test piece. To-day the elements affecting the strength of wood are much better known. While we still obtain the same wide range of values we are able to assign causes therefor.

It is safe to say that the time is coming when, by occular inspection, an expert will be able to predict the strength of a piece of wood more accurately than can be done by the same method with iron or steel. Rules of inspection may then be formulated which, if carefully followed, will yield timber of comparatively uniform strength. But even with the knowledge we now have it is possible to improve greatly upon the present practice.

With this object in view a query sheet was sent to about thirty of the principal railroad companies of the United States for the purpose of determining what the prevailing practice now is. Answers were received from twenty of these companies, and serviceable information from fifteen, ten of which represented 500 miles of trestle. The other five did not report mileage. This information has been compiled in Tables I and II.

DATA FOR TRESTLE CONSTRUCTION.

PRESENT PRACTICE.

Table I gives the different species now employed in the various parts of these structures and a mean estimate of the length of life of each. These separate estimates, however, were very erratic, in many cases being little better than a guess, so that the mean given in the table is by no means reliable.

This lack of information is scarcely less remarkable than it is unfortunate. Although for more than fifty years railroad companies have been using timber, no accurate, classified knowledge exists as to its length of life; yet this could be easily obtained if each member of a trestle were given a number, as is done in iron structures. The length of life of timber is, of course, not an exact quantity, being a function not only of the various conditions of use, but also those of growth and treatment previous to use. For a given locality and treatment the length of life of originally sound timber of a given species should not exhibit such a remarkable difference in durability as indicated on the separate returns from the railroad companies.

From an examination of the mean value given at the bottom of Table I we see that, in general, the piles and posts outlast the stringers and caps by from one to two years. This is reasonable and undoubtedly correct in kind; probably an underestimate in degree.

TABLE I.—*Species of timber employed in the construction of wooden trestle bridges.*

[Compiled from reports from fifteen railroads of the United States.]

Species used.	Mean length of life.	Per cent of reported roads using this species.
PILES.		
	Years.	
White oak	10	70
Burr oak	....	12
Red cedar	16	31
Redwood	12	6
Longleaf pine	9	25
Oregon or Douglas fir	8-9	12
Red cypress	7	6
White pine	7	6
Norway pine	7	6
Sugar pine	....	6
Red spruce	....	6
Mean	9.6	....
STRINGERS.		
Douglas or Oregon fir	10	44
Longleaf pine	8	56
Shortleaf pine	6	6
White pine	8	44
Norway pine	8	6
Red cypress	7	6
Mean	7.8	....
CAPS.		
	Years.	
Douglas or Oregon fir	10	25
Longleaf pine	8	44
Shortleaf pine	6	6
White pine	8	38
Norway pine	8	12
Colorado pine	8	6
Red cypress	10	19
Red cedar	11	6
White oak	8	19
Redwood	12	6
Mean	8.9	....
POSTS.		
Douglas or Oregon fir	12	25
Longleaf pine	10	56
Shortleaf pine	6	6
White pine	10	38
Norway pine	9	12
Red cypress	9	12
Red cedar	11	6
White oak	9	19
Redwood	12	6
Mean	9.8	....

TABLE II.—*Showing the range in values of safe unit stresses as reported by fifteen railroad companies of the United States.*

[The "factor of safety" given below is not what was reported by these companies, their values ranging from five to twelve, but what actually exists according to values based on the tests of the Forestry Division as given in Table IV.]

Species.	Modulus of strength at rupture per square inch.	Modulus of elasticity per square inch.	Crushing strength endwise per square inch.	Crushing strength across the grain per square inch.
	Pounds.	*Pounds.*	*Pounds.*	*Pounds.*
Longleaf pine (*Pinus palustris*)	1,000-5,000	....	*830-1,700	650-1,000
Shortleaf pine (*Pinus echinata*)	....	....	....	....
White pine (*Pinus strobus*)	500-1,000	....	600-1,000	400- 420
Norway pine (*Pinus resinosa*)	....	....	650-2,100	350
Colorado pine (*Pinus ponderosa*)	1,360	....	....	....
Douglas fir (*Pseudotsuga douglasii*)	500-2,160	....	600-1,000	120- 420
Redwood (*Sequoia sempervirens*)	600	....	300	60
Red cedar (*Juniperus virginiana*)	1,750	....	1,000	530
Bald cypress (*Taxodium distichum*)	1,600	....	1,000	260
White oak (*Quercus alba*)	3,300	....	1,400	600
Factor of safety	1.5-13.0	....	2.6-25.0	0.6-5.7

*The reporter of this value gave 10,000 pounds per square inch as the modulus of rupture, and said in designing he was accustomed to use one-half the amount.

TABLE II shows the range in values for the safe unit stresses as reported by these railroad companies. This range is best appreciated by an inspection of the range in value of the factor of safety for the different factors of strength, given at the bottom of the table, which is based upon the safe-load values given in Table IV. Supposing these

latter values to be correct, we find that where a factor of safety of say 5 to 12 was supposed to exist, this quantity has a range of from 0.6 to 25. The former value occurs for the crushing strength across the grain by which the bearing area on the cap of a trestle bent should be proportioned. Less seems to be known about this quality of timber than any of the others, though very little is known of any of them. Two companies reported equal values for the crushing strength endwise and across the grain.

Most of the safe loads assumed by these companies for this latter factor would, if actually employed, be dangerously unsafe. Fortunately, perhaps, trestles have not been designed according to these or any other safe loads, but according to standard sizes which have proved themselves capable of doing the required duty—at least for a time. But even these furnish an altogether insufficient factor of safety in many cases, while in others it becomes five times as large as necessary. No values whatever were given for the modulus of elasticity.

TABLE III.—*Showing the great range in value of the factor of safety for the different portions of timber trestles, according to the practice now prevailing.*

[Span=14 feet; length post=12 feet; posts, caps, and sills, all 12 inches square. Load=100-ton consolidation engine of the P. R. R. Maximum moment=98,600 foot-pounds on one rail; maximum shear=36,000 pounds on one rail; maximum bent load=91,800 pounds on both rails.]

Species.	Stringers, all 16 inches high. Factor of safety.		Caps. Factor of safety for bearing value under stringers of—							Caps.	Factor of safety for posts.
	In cross breaking.	For bearing on cap.	Longleaf pine.	Shortleaf pine.	White pine.	Norway pine.	Colorado pine.	Douglas fir.	Cypress.	Factor of safety for bearing value under post.	
Longleaf pine	5	1.92	1.92	2.28	3.38	2.74	3.04	2.25	2.97	3.03	24.4
Shortleaf pine	5	2.30	1.92	2.28	3.38	2.74	3.04	2.25	2.97	3.03	20.5
White pine	5	2.32	1.32	1.56	2.32	1.87	2.08	1.54	2.04	2.07	17.1
Norway pine	5	1.82	1.28	1.52	2.25	1.82	2.02	1.50	1.98	2.02	18.4
Colorado pine	5	2.55	1.61	1.92	2.83	2.30	2.55	1.89	2.49	2.54	15.3
Douglas fir	5	1.76	1.50	1.78	2.63	2.13	2.37	1.75	2.32	2.35	21.4
Redwood	......	......	1.03	1.23	1.81	1.46	1.63	1.21	1.59	1.62	15.8
Cedar	......	......	2.24	2.67	3.94	3.18	3.54	2.62	3.46	3.52	17.1
Cypress	5	1.66	1.07	1.28	1.89	1.53	1.70	1.26	1.66	1.69	16.4
White oak	......	......	3.58	4.26	6.30	5.10	5.66	4.20	5.54	5.63	19.5

This table gives, for a set of conditions representative of the present practice, the factors of safety obtaining for the posts and for the bearing value of stringers on caps and caps on posts, using the safe loads given in Table IV and a uniform height of stringer of 16 inches. That is to say, the load assumed and the dimensions of caps and posts and height of stringer are supposed to represent the average conditions in practice. The necessary width of stringer, however, has been determined by using the safe loads recommended in Table IV. While in some cases this resulting width, from which the bearing area is computed, will be too small to represent the average practice, in other

cases it will be too large, so that it is thought to be fairly representative thereof. It is also assumed that the stringers are butting, each having but 6 inches bearing at each end on the cap.

A factor of safety of 3 (see discussion of Table IV) is necessary and sufficient for the proportioning of areas subject to load across the grain.

From an inspection of Table III, under the head of "Stringers"—the latter being correctly designed to resist the bending moment—we see that stringers of none of these species would have sufficient bearing area upon a 12 by 12 inch cap. Colorado pine comes nearest, with a factor of safety of 2.55. The resistance of the cap under the stringer depends upon the kind of stringer used. For example, with a white-pine cap and Douglas-fir stringers, the cap has a factor of safety of 1.54. The last column but one in the table gives the factor of safety for the bearing value of cap on the post.

There are only four species, i. e., longleaf and shortleaf pine, cedar, and oak, that have sufficient strength in crushing across the grain to enable the cap to resist the thrust of the post. Oak is the only species that has sufficient strength to enable the cap to resist the pressure of all kinds of stringers.

There has been no such parsimony exercised in the design of the posts, however. The factors of safety here range from 15.8 to 24.4.

Now, although the stringers in cross breaking have a factor of safety of 5, and the posts have a factor of safety of 20, the structure as a whole has a factor of safety of only 2, approximately.* Since these values are intended to represent only the average condition in practice, they are very unlikely to represent any actual condition, being functions of such variable quantities as the proof load, length and height of span, height of stringer, etc.

They serve to show, however, the wholly inefficient allowance that has been made in the bearing area, and it is probable that in nine out of ten cases no attempt has been made to proportion this surface according to conditions in hand.

RECOMMENDED PRACTICE.

Since the strength of timber varies very greatly with the moisture contents (see Bulletin 8 of the Forestry Division), the economical designing of such structures will necessitate their being separated into groups according to the maximum moisture contents in use.

MOISTURE CLASSIFICATION.

Class A (moisture contents, 18 per cent).—Structures freely exposed to the weather, such as railway trestles, uncovered bridges, etc.

*This is based upon the assumption that it is dangerous to strain a cap or stringer in crushing across the grain more than 3 per cent of the height. (See page 14.) While this amount of crushing is not a failure in the sense of collapse of the structure, yet unless the piece is soon taken out and replaced, the structure as a whole will be in danger.

Class B (moisture contents, 15 per cent).—Structures under roof but without side shelter, freely exposed to outside air, but protected from rain, such as roof trusses of open shops and sheds, covered bridges over streams, etc.

Class C (moisture contents, 12 per cent).—Structures in buildings unheated, but more or less protected from outside air, such as roof trusses or barns, inclosed shops and sheds, etc.

Class D (moisture contents, 10 per cent).—Structures in buildings at all times protected from the outside air, heated in the winter, such as roof trusses in houses, halls, churches, etc.

The following tables of safe loads have all been made out for Class A, with the intention of making them applicable to bridge-trestle construction. To make these applicable to the other classes make the following modifications:

For longleaf pine add to all the values given in the tables, except those for moduli of elasticity, tension, and shearing, for Class B, 15 per cent; for Class C, 40 per cent, and for Class D, 55 per cent. For the other species add to these values, for Class B, 8 per cent; for Class C, 18 per cent, and for Class D, 25 per cent.

For the modulus of elasticity add only one-half of the above percentages. For tension and shearing use the tabular values— whatever the percentage of moisture.

For longleaf and shortleaf pine these modifications are quite correct, the percentage of increase of strength of the former being about twice as great as that for the latter between the green and dry condition. This percentage of increase is not so well known for the other species, but tests that have been made indicate a percentage of increase at least as large as for shortleaf pine. Until further tests have been made, therefore, the modifications given above may safely be used.

NOTE.—The reductions for moisture as given above in the case of longleaf pine appear somewhat at variance with results obtained since. In the case of other species they rest on assumption, for which experimental data are still largely lacking; it will, therefore, be proper to use the same with caution.

The moisture condition at 18 per cent is one difficult to obtain under natural conditions; it would, therefore, have been more desirable if the author had started from the green condition, which is fixed. The following values for green condition are here added for the four Southern pines, on which alone the Forestry Division has reliable data:

	Moisture condition.	Cuban pine.	Longleaf.	Loblolly.	Shortleaf.	Average change.
	Per cent.					
Transverse strength or modulus of rupture, green	33	6,150	6,200	5,830	5,230	....
Compression endwise, green	33	4,150	3,660	3,430	3,360	....
Relative strength as a mean of transverse and compression:						
Green	33	100	100	100	100	100
Half dry	20	125	119	122	120	122
Yard dry	15	149	148	147	138	146
Room dry	10	182	194	187	165	182

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Table IV is a table of safe unit stresses * of the various kinds for the materials employed in the construction of timber trestles. The "safe unit stress" is equal to the ultimate strength, as determined from the test, divided by a quantity which is called the factor of safety.

SAFE UNIT STRESSES.

TABLE IV.—*Safe unit stresses at 18 per cent moisture.*

[The values marked "D" were obtained from experiments made by the Forestry Division. The other values were obtained from various sources, chiefly the Tenth Census report, but so modified as to give results comparable with Forestry Division values. To arrive at true average values of strength, multiply safe loads by factor of safety given in each column. The values for resilience and tensile strength are the *ultimate* values. The former is practically never used in designing. The latter is a factor impossible to develop in practice, since the piece will always fail in some other way, usually by shearing. (See descriptive text.)]

Species.	Modulus of strength at rupture per square inch.	Modulus of elasticity per square inch.	Elastic resilience per cubic inch.	Crushing strength endwise per square inch.	Crushing strength across the grain per square inch.	Tensile strength per square inch.	Shearing strength per square inch.
	Pounds.	*Pounds.*	*Pounds.*	*Pounds.*	*Pounds.*	*Pounds.*	*Pounds.*
Longleaf pine (*Pinus palustris*), D	1,550	720,000	1.30	1,000	215	12,000	125
Shortleaf pine (*Pinus echinata*), D	1,300	600,000	1.30	840	215	9,000	100
White pine (*Pinus strobus*)	880	435,000	1.00	700	147	7,000	75
Norway pine (*Pinus resinosa*)	1,090	566,000	....	760	148	....	....
Colorado pine (*Pinus ponderosa*)	980	444,000	....	630	180	....	....
Douglas fir (*Pseudotsuga douglasii*)	1,320	690,000	....	880	167	....	....
Redwood (*Sequoia sempervirens*)	1,440	226,000	....	650	115	....	....
Red cedar (*Juniperus virginiana*)	1,000	335,000	....	700	250	....	....
Bald cypress (*Taxodium distichum*), D	1,000	450,000	1.10	675	120	6,000	60
White oak (*Quercus alba*), D	1,200	550,000	1.25	800	100	10,000	200
Factor of safety	5	2	1	5	3	1	4

In designing add one-half inch to each dimension obtained by use of above table to allow for weathering.

The values marked "D" in the table were obtained from tests made by the Forestry Division and are considered quite reliable, especially those for longleaf and shortleaf pine. These are, as indicated above, for a moisture of 18 per cent, representing a half dry condition, and were taken from a minimum moisture curve † which represented the average strength of the lowest 10 per cent of the nondefective pieces tested. This curve gives values from 15 to 20 per cent less than the mean values obtained for the species, and material of this strength can readily be obtained even for full-sized beams and columns by an inspector of average intelligence.

The other values were obtained from various sources, chief of which was the Tenth Census report. They were not taken as there given, however, but modified in the following manner to make them comparable with the Forestry Division values: The mean of the values given in the census report for the "D" timbers in Table IV were compared with the mean of these same values there given; the ratio of these two was

* For a description of these stresses see page 29.

† See Bulletin 8.

used as a factor of reduction for applying to the census report values for the species upon which the Forestry Division has so far made no tests.

The information for these species is very meager, but the values given are considered safe, though probably not as economical as they might be if more extensive tests had been made.

As will be seen from an inspection of Table IV, the factor of safety is not a constant quantity, but ranges from 2 to 5.

In general, any composite structure should be of equal strength in all of its parts. This does not mean that they should all have the same factor of safety.

This factor is a function of three things:

(1) The importance of the piece in the structure.

(2) The amount of ignorance as to the strength of the material.

(3) The amount of ignorance as to the amount of the imposed load.

In many cases the failure of one piece will not endanger the structure as a whole. In other cases, even if the whole structure is wrecked, no serious calamity results. In these cases a small factor of safety may be used.

The values given for the modulus of elasticity for all the species except redwood and cedar will give, for the average condition,* a deflection equal to about one two-hundredths of the span, which has been assumed as the maximum allowable. This is about equal to a factor of safety of 5 on the total deflection at rupture. The exceptions to this are the two species above mentioned, but these are not used for beams.

The crushing strength across the grain in Table IV is based upon a crushing of 3 per cent of the cross-sectional height of the piece. This point may be compared to the elastic limit † in the cross-breaking tests. While absolute failure does not occur at this point, yet it is a point beyond which it is unsafe to go. The point of absolute failure in this test is, more or less, an imaginary point, and the above percentage of crushing has been selected as an arbitrary representative thereof. As failure does not occur here, however, a factor of safety of 3 is deemed necessary and sufficient for this kind of a load.

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* See page 13. † See Bulletin 8, page 23.

Table V.—*Safe loads, in pounds, for longleaf-pine (Pinus palustris) beams uniformly loaded, and 1 inch in width.*

[For other species multiply these loads by the ratio of the safe modulus of rupture for the species as given in Table IV, divided by 1,550. Thus, for shortleaf pine these loads should be multiplied by $\frac{1,300}{1,550}$. For other widths, multiply by the width. For single load in middle of beam, divide these loads by 2. To avoid danger of shearing along the neutral plane, the ratio of length to height of uniformly loaded beam should be at least 10; for beam with single load in middle, not less than 8.]

Length of beam.	Height of beam in inches.												
	6	7	8	9	10	11	12	13	14	15	16	17	18
Feet.													
4	1,550	......	......	......	......	......	......	......	......	......	......	......	......
5	1,240	1,687	......	......	......	......	......	......	......	......	......	......	......
6	1,033	1,406	1,837	2,325	......	......	......	......	......	......	......	......	......
7	885	1,206	2,574	1,992	2,460	......	......	......	......	......	......	......	......
8	775	1,054	1,376	1,742	2,150	2,603	3,100	......	......	......	......	......	......
9	688	935	1,224	1,550	1,914	2,316	2,755	3,235	......	......	......	......	......
10	620	844	1,101	1,394	1,720	2,083	2,480	2,910	3,370	3,875	......	......	......
11	562	768	1,000	1,267	1,564	1,893	2,253	2,645	3,068	3,520	4,010	......	......
12	517	702	917	1,160	1,434	1,736	2,067	2,426	2,812	3,226	3,673	4,150	4,650
13	477	649	847	1,072	1,322	1,601	1,906	2,238	2,595	2,980	3,390	3,825	4,285
14	442	603	787	995	1,230	1,488	1,770	2,077	2,408	2,763	3,145	3,550	3,980
15	413	562	733	930	1,147	1,387	1,653	1,940	2,250	2,580	2,940	3,320	3,720
16	386	527	688	872	1,074	1,302	1,550	1,818	2,109	2,420	2,753	3,108	3,485
17	364	496	647	820	1,013	1,226	1,459	1,711	1,984	2,278	2,590	2,925	3,280
18	343	469	612	775	956	1,157	1,379	1,619	1,878	2,152	2,450	2,762	3,100
19	327	445	580	733	905	1,097	1,305	1,533	1,777	2,039	2,320	2,618	2,933
20	310	422	550	696	860	1,041	1,240	1,455	1,689	1,940	2,206	2,490	2,790
21	296	401	525	663	820	992	1,181	1,387	1,609	1,845	2,100	2,370	2,656
22	281	384	500	635	783	946	1,129	1,325	1,536	1,762	2,005	2,260	2,535
23	269	366	479	607	748	905	1,079	1,264	1,467	1,684	1,919	2,161	2,424
24	258	352	458	581	717	868	1,033	1,212	1,408	1,616	1,837	2,072	2,321
25	248	337	440	558	688	833	992	1,164	1,350	1,550	1,762	1,990	2,231
26	238	324	424	535	662	802	954	1,120	1,299	1,491	1,697	1,913	2,142
27	230	312	407	517	637	773	920	1,079	1,250	1,434	1,635	1,842	2,066
28	221	302	393	498	613	745	885	1,040	1,206	1,384	1,575	1,777	1,992
29	213	292	380	482	593	718	855	1,005	1,163	1,338	1,521	1,718	1,925
30	203	281	368	465	575	694	827	968	1,127	1,292	1,470	1,660	1,860

Safe-load formula.

Uniform load, $S=\frac{4\,R\,b\,h^2}{3\,l}$

Single load in middle, $S=\frac{2\,R\,b\,h^2}{3\,l}$

where

R=modulus of rupture. (See Table IV.)
S=safe load in pounds.
b=breadth of beam in inches.
h=height of beam in inches.
l=length of beam in inches.

Deflection formula.

Uniform load, $\triangle=\frac{5\,R\,l^2}{24\,E\,h}$

Single load in middle, $\triangle=\frac{R\,l^2}{6\,E\,h}$

$\triangle$=deflection at center in inches.
E=mouulus of elasticity. (See Table IV.)
Other quantities same as for safe-load formula.

Table V gives the safe load on longleaf-pine beams, uniformly loaded and 1 inch thick. By a uniform load is meant a constant load per running foot of beam.

TABLE VI.—*Safe load on square columns of longleaf pine* (*Pinus palustris*).

[Load given in tons of 2,000 pounds.]

Length in feet.	Side in inches.											
	4	5	6	7	8	9	10	11	12	13	14	15
2	7.65	12.2	17.7	24.2	31.7	......	......	......	......	......	......	......
4	6.90	11.2	16.7	23.2	30.6	38.2	48.6	58.9	70.5	......	......	......
6	5.99	10.2	15.5	21.8	29.3	37.6	47	57.4	68.8	81.4	94.9	109.2
8	5.18	9.1	14.1	20.3	27.5	35.7	45	55.2	66.9	79.4	92.7	107
10	4.50	8.08	12.9	18.7	25.8	34	43	53.2	64.4	77	90	104.5
12	3.90	7.21	11.7	17.3	23.9	31.8	40.8	50.7	62	74	86.9	101.4
14	3.43	6.46	10.6	15.8	22.3	29.7	38.6	48.3	59.3	71	84.3	98.4
16	3.05	5.81	9.6	14.6	20.7	28.2	36.5	46	56.6	68.4	81.3	95.1
18	......	5.22	8.81	13.8	19.3	26.2	34.3	43.6	53.5	65.4	78.4	91.6
20	......	4.75	8.10	12.4	18	24.7	32.3	41.2	51.4	62.6	75.1	88.4
22	......	......	7.41	11.6	16.8	23.4	30.6	39.1	49	59.9	72	85.2
24	......	......	6.84	10.7	15.6	21.8	28.9	37.1	46.6	57.2	69	82
26	......	......	......	10	14.6	20.4	27.3	35.3	44.4	54.5	66.2	78.7
28	......	......	......	9.33	13.7	19.4	25.9	33.7	42.3	52.3	63.4	75.5
30	......	......	......	......	12.9	18.2	24.4	31.7	40.6	50.3	60.8	72.6
32	......	......	......	......	12.2	17.1	23.3	30.3	38.4	48	58.6	70.1
34	......	......	......	......	......	16.3	22	29	36.9	45.8	56	67.5
36	......	......	......	......	......	15.4	20.9	27.6	35.2	44	54	65
38	......	......	......	......	......	14.8	20	26.1	33.7	42	52.3	62.7
40	......	......	......	......	......	......	19	25	32.3	40.6	49.7	60.1
42	......	......	......	......	......	......	18.1	24	30.8	38.6	47.9	58.3
44	......	......	......	......	......	......	......	23	29.6	37.4	46.2	56
46	......	......	......	......	......	......	......	22	28.6	36	44.5	54
48	......	......	......	......	......	......	......	......	27.4	34.6	52.8	52.5
50	......	......	......	......	......	......	......	......	26.3	33.4	41.6	50.6

Table VI gives the total safe load for square columns of longleaf pine in tons of 2,000 pounds per square inch.

In designing, one-half inch is to be added to each cross-sectional dimension obtained from these two tables to allow for weathering.

The ordinary column formulæ take this form:

$$f = \frac{F}{1 + \frac{1}{a}\left(\frac{l}{d}\right)^2}$$

f = allowable working stress per square inch for the long column.
F = allowable working stress per square inch for the short column.
a = constant, for square-ended columns, usually taken at from 500 to 550.
l = length in inches.
d = least cross-sectional dimension in inches.

From experiments made upon about 50 columns by this division, the following formula may be deduced, which closely coincides with the mean of the plotted points as shown on fig. 1.

$$f = \frac{F}{1 + \frac{1}{700 + 15\left(\frac{l}{d}\right)}\left(\frac{l}{d}\right)^2}$$

That is to say, that a, instead of being a constant, as usually assumed, has been found to be a rectilinear function of $\frac{l}{d}$

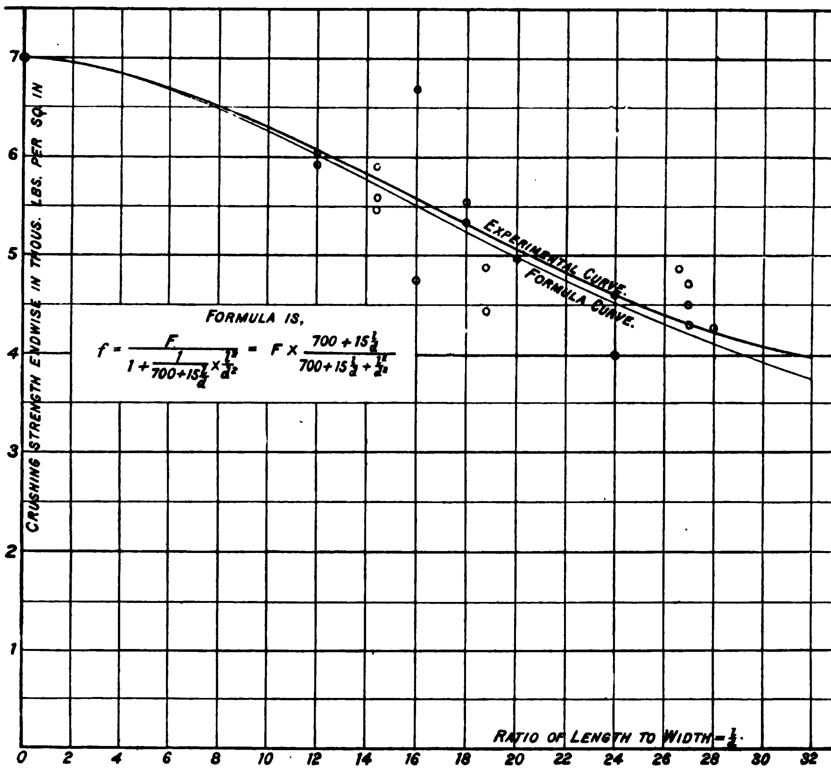


Fig. 1.—Diagram showing the relation between the coefficient of crushing endwise strength and the ratio of length to least width of solid timber columns of longleaf pine.

This equation, transformed, becomes

$$f = \mathrm{F} + \frac{700+15c}{700+15c+c^2}, \text{ where } c = \frac{l}{d}$$

The values in Table VI have been computed by this formula.

Table VII.—*Giving length of grip at end bearing required for uniformly loaded longleaf-pine (Pinus palustris) beams of varying heights and lengths of span.*

Formula: $c = \frac{2 R h^2}{3 S l}$ or for given species $c = K \frac{h^2}{l}$

where

c = required length of grip in inches.
R = safe modulus of rupture. (See Table IV.)
S = safe crushing strength across the grain. (See Table IV.)
h = height of beam in inches.
l = length of span in inches.
K = a constant, depending upon the material used.

For longleaf pine K=4.8; shortleaf, 4.0; white pine, 4.0; Norway pine, 5.0; Douglas fir, 5.28; bald cypress, 5.55.

To obtain the tabular values for other species multiply the value for the longleaf pine given in the table by the ratio of $\frac{K' \text{ (for the species)}}{K \text{ (for longleaf pine)}}$.

Height of beam in inches.	Length of span in feet.											
	6	7	8	9	10	11	12	13	14	15	16	17
6	2.4	2.1	1.8	1.6	1.4	1.3	1.2	1.1	1.0	1	0.9	0.8
7	3.3	2.8	2.4	2.2	2	1.8	1.6	1.5	1.4	1.3	1.2	1.2
8	4.3	3.7	3.2	2.8	2.6	2.3	2.1	2	1.8	1.7	1.6	1.5
9	5.4	4.6	4.1	3.6	3.2	2.9	2.7	2.5	2.3	2.2	2	1.9
10	6.7	5.7	5	4.4	4	3.6	3.4	3.1	2.9	2.7	2.5	2.4
11	8.1	6.9	6.1	5.4	4.8	4.4	4	3.7	3.5	3.2	3	2.8
12	9.6	8.2	7.2	6.4	5.8	5.2	4.8	4.4	4.1	3.8	3.6	3.4
13	11.3	9.7	8.4	7.5	6.8	6.1	5.6	5.2	4.8	4.5	4.2	4
14	13.1	11.2	9.8	8.7	7.8	7.1	6.5	6	5.6	5.2	4.9	4.6
15	15	12.8	11.2	10	9	8.2	7.5	6.9	6.4	6	5.6	5.3
16	17.1	14.6	12.8	11.4	10.2	9.3	8.5	7.9	7.3	6.8	6.4	6
17	19.6	16.5	14.4	12.8	11.6	10.5	9.6	8.9	8.3	7.7	7.2	6.8
18	21.6	18.5	16.2	14.4	13	11.8	10.8	10	9.3	8.6	8.1	7.6

Height of beam in inches.	Length of span in feet.										
	18	19	20	21	22	23	24	25	26	27	28
6	0.8	0.8	0.7	0.7	0.7	0.6	0.6	0.6	0.6	0.5	0.5
7	1.1	1	1	.9	.9	.9	.8	.8	.8	.7	.7
8	1.4	1.3	1.3	1.2	1.2	1.1	1.1	1	1	.9	.9
9	1.8	1.7	1.6	1.5	1.5	1.4	1.3	1.3	1.2	1.2	1.2
10	2.2	2.1	2	1.9	1.8	1.7	1.7	1.6	1.5	1.5	1.4
11	2.7	2.5	2.4	2.3	2.2	2.1	2	1.9	1.9	1.8	1.7
12	3.2	3	2.8	2.7	2.6	2.5	2.4	2.3	2.2	2.1	2.1
13	3.8	3.6	3.4	3.2	3.1	2.9	2.8	2.7	2.6	2.5	2.4
14	4.4	4.1	3.9	3.7	3.6	3.4	3.3	3.1	3	2.9	2.8
15	5	4.7	4.5	4.3	4.1	3.9	3.7	3.6	3.5	3.3	3.2
16	5.7	5.4	5.1	4.9	4.6	4.4	4.3	4.1	3.9	3.8	3.7
17	6.4	6.1	5.8	5.5	5.3	5	4.8	4.6	4.4	4.3	4.1
18	7.2	6.8	6.5	6.2	5.9	5.6	5.4	5.2	5	4.8	4.6

For beams under concentrated loads, such as trestle stringers, add 30 per cent to above values.
For single concentrated load in middle of beam divide these values by 2.

Table VII gives the length of grip at end bearing required for uniformly loaded longleaf-pine beams of varying heights and lengths of span.

The formula $c = \frac{2 R h^2}{3 S l}$ has been obtained in the following manner:

$S = \frac{W'}{2 c b}$ or $W' = 2 S c b$ (see Table VII for definition of quantities)

and R for uniform load $= \frac{3 W'' l}{4 b h^2}$ or $W'' = \frac{4 R b h^2}{3 l}$.

Where W' = total load on beam at time the end bearing has its proof load.

W'' = total load on beam at time the extreme fiber has its proof load.

Making $W' = W''$ we have

$$2\,S c b = \frac{4\,R b h^2}{3\,l}$$

or

$$c = \frac{2\,R h^2}{3\,S l} \text{ for uniformly loaded beams, or}$$

$$c = \frac{R h^2}{3\,S l} \text{ for concentrated loads at middle.}$$

Problem: What is the length of grip necessary at the end of a white-pine stringer 14 feet long and 16 inches high?

The table gives for these dimensions 7.3 inches as the grip required for a longleaf-pine beam uniformly loaded.

The value of $\frac{K'}{K}$ in this case is equal to $\frac{4.0}{4.8} = \frac{1}{1.2}$

Also we must add 30 per cent on account of the difference in the character of the load. Hence we have $7.3 \times \frac{1}{1.2} \times 1.3 = 7.9$ inches.

INSPECTION.

Scientific inspection of timber requires a knowledge of the elements affecting the strength thereof.

The following are the principal elements: Moisture condition; weight per cubic foot; size of piece: position in tree; defects, such as knots, cross-graining, ring shakes, and season checks; anatomical structure; character of the secretions, such as resin, etc.; method of treatment previous to use.

Moisture condition.—This is the chief element affecting the strength of timber. The strength of a thoroughly seasoned piece is from 50 to 100 per cent more than that of a green piece. To say that a certain piece of timber has a strength of a certain amount is of no service whatever in determining its relative strength unless the moisture condition is specified.

Weight.—In material of the same species, having practically the same anatomical structure and character of secretions, the strength varies directly as the weight for the same condition of moisture.

Size of piece.—This has some effect, but it is much less than has been generally supposed. The chief difficulty is in the seasoning of large pieces. If this is done carefully no allowance need be made in the safe loads given on account of size, except for more frequent defects.

Position in tree.—The strength varies with the position in tree. In old trees of pine, from 150 to 200 years of age, the strongest portion of the butt log will be at about one-half the radius from the center. As we go higher in the tree, the central part, though weaker than in the butt log, becomes the strongest portion of the cross section.

Defects.—Large knots should not be allowed to come at the middle of a beam, either on top or bottom, as they are a source of weakness in compression as well as in tension, though not quite to the same extent.

The fibers around a knot run nearly at right angles to the axis of the tree, so that in a compression test these fibers are subject to a crushing strength across the grain, in which direction they are very weak.

Season cracks on top of a beam have little effect upon the strength, except as they may collect water and start rot. Those, however, on the side of a beam near the neutral plane, which for timber is usually a little below the middle, are very injurious, as they greatly increase the liability to shear along this plane.

Wooden beams, as ordinarily employed, are more apt to fail in this manner than in any other way. By putting a bolt in each end of large beams, thus firmly holding the top and bottom portions together, this danger could be largely avoided. Ring shakes are also prime causes of shearing along the neutral plane.

Anatomical structure.—This has, of course, great influence upon the strength of timber, and to it is largely due the difference in strength between the different species.

But little is known about this subject as yet, except in a general way, as, for example, regarding pine and oak.

The present time is a little premature for the formulation of full rules for scientific inspection of timber. We only know that for strength we require dry timber, and for a given species the heavier the stronger.

For the present it is impossible to evaluate the effect of knots and other defects, but we should guard against them as far as possible.

The safe loads given in the tables herein will then be perfectly safe, and apply to all sizes.

METHODS OF DESIGNING.

PRESENT PRACTICE.

Many of the railroad companies now use a safe load of 1,000 pounds per square inch for the modulus of rupture for longleaf-pine stringers. The caps, sills, and posts are usually 12 by 12 inches, irrespective of load.

Fig. 2 represents a common type of construction designed by the above considerations and for the same conditions given in Table III.

The formula for bending is*

$$M = \frac{R b h^2}{6}$$

M=bending moment in pounds per square inch.

R=safe load on extreme fiber in pounds per square inch.

b=breadth of beam in inches.

h=height of beam in inches.

Transformed, this becomes

$$b = \frac{6 M}{R h^2}$$

*See Appendix 1.

Substituting the values for these quantities, we have

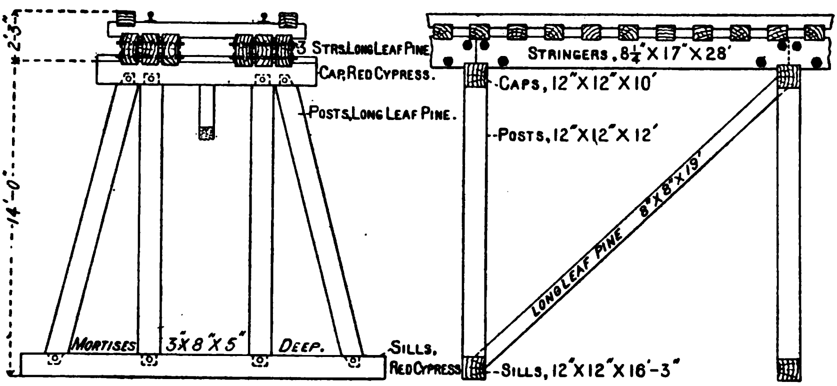
$$b = \frac{6 \times 98{,}600 \times 12}{1{,}000 \times 289}$$

where the height is assumed for trial.

This will make three stringers under each rail, 8½ by 17 inches in cross section, posts, caps, and sills all being 12 by 12 inches in cross section.

The following factors of safety are indicated by their practice:

Stringers in cross breaking	7.6
Stringers in deflection $\frac{1}{250}$ span	3.1
Stringers in bearing value	2.7
Cap, bearing value under stringers	1.5
Cap, bearing value under posts	1.9
Posts crushing endwise	[illegible].6



SCALE: ⅛ INCH = 1. FOOT.

FIG. 2.—Example of present practice of designing.

Bill of material.

TIMBER, EXCLUSIVE OF TIES AND GUARD RAIL.

Species.	Used for—	Size.	Number feet, B.M.	Cost per thousand.	Total cost.
Longleaf pine	Stringers	6 pieces, 8½″×17″×14′ in 28′ lengths	982	$13.00	$12.80
Red cypress	Caps	1 piece, 12″×12″×10′	120	11.00	1.32
Longleaf pine	Posts	4 pieces, 12″×12″×13′ 6″	648	8.00	5.18
Red cypress	Sills	1 piece, 12″×12″×16′ 3″	195	11.00	2.14
Longleaf pine	S. braces	1 piece, 8″×8″×19′	101	8.00	.81
			2,046		22.25
		Cost of iron			2.86
		Total cost of panel			25.11

IRON.

Size.	Number of pounds.	Cost per pound.	Total cost.
		Cents.	
Bolts, 8 pieces, ¾″×32½″, at 5.20	41.6	4	$1.66
Driftbolts, 2 pieces, 1″×23″, at 7.5	15	4	.60
Washers, O. G., 16 pieces, 3″ diameter, at 1.25	24	2½	.60
Total	80.6		2.86

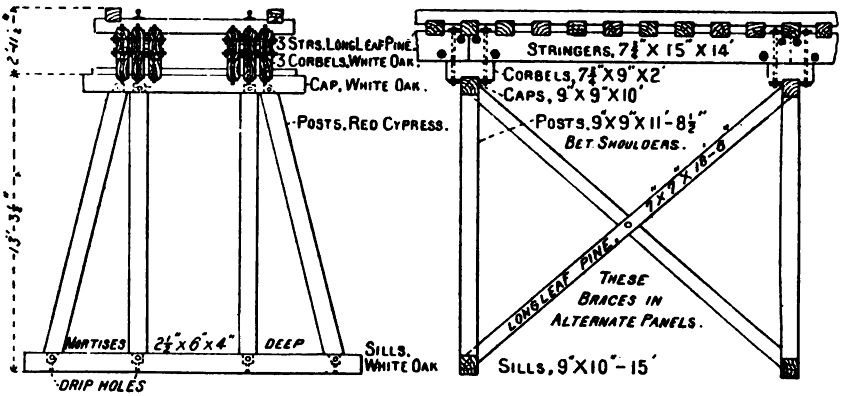
Figs. 3 and 4 show designs recommended by the Forestry Division, that in fig. 3 being preferred, though slightly more expensive than that in fig. 4. These diagrams illustrate an important point, i. e., that the economical design of timber structures requires the *judicious employment of different species as well as different sizes*, in the same.

RECOMMENDED PRACTICE.

With corbels:

Factors of safety.

Stringers in cross breaking	5 +
Stringers in deflection $\frac{1}{200}$ span	2 +
Stringers in end bearing	4 +
Cap in bearing value	3 +
Posts in endwise crushing	7.4+



SCALE: $\frac{1}{8}$ INCH = 1 FOOT.

FIG. 3.—Example of proposed practice with corbels.

Bill of material.

TIMBER, EXCLUSIVE OF TIES AND GUARD RAIL.

Species.	Used for—	Size.	Number feet, B.M.	Cost per thousand.	Total cost.
Longleaf pine	Stringers	6 pieces, 7¾″×15″×14′, in 14′ lengths.	815	$11.00	$8.96
White oak	Caps	1 piece, 9″×9″×10	68	11.00	.75
Red cypress	Posts	4 pieces, 9″×9″×13′	351	8.00	2.81
White oak	Sills	1 piece, 9″×10″×15′	112	11.00	1.23
Longleaf pine	S. braces	1 piece, 9″×7″×18′ 6″	76	8.00	.61
White oak	Corbels	6 pieces, 7¾″×9″×2′	58	8.00	.46
			1,480		14.82
		Cost of iron			5.31
		Total cost of panel			20.13

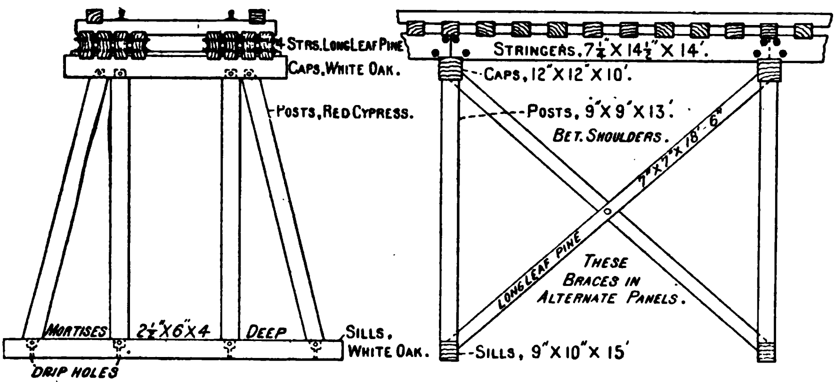
IRON.

Size.	Number of pounds.	Cost per pound.	Total cost.
		Cents.	
Bolts (stringers), 8 pieces, ¾″×31″, at 4.96	39.7	4	$1.59
Bolts (corbels), 12 pieces, ¾″×28″ at 4.48	53.8	4	2.15
Driftbolts, 2 pieces, 1″×12″ at 4.00	8	4	.32
Washers, O. G., 40 pieces, 3″ diameter, 1.25	50	2½	1.25
Total	151.5		5.31

Without corbels:

Factors of safety.

Stringers in cross breaking	5 +
Stringers in deflection 1/200 span	2 +
Stringers in end bearing	3 +
Cap in bearing value	3 +
Posts in endwise crushing	7.4+



SCALE: 1/4 INCH = 1 FOOT.

FIG. 4.—Example of proposed practice without corbels.

Bill of material.

TIMBER, EXCLUSIVE OF TIES AND GUARD RAIL.

Species.	Used for—	Size.	Number feet, B.M.	Cost per thousand.	Total cost.
Longleaf pine	Stringers	8 pieces, 7½″×14.5″×14′ long	982	$10.50	$10.31
White oak	Caps	1 piece, 12″×12″×10′	120	11.00	1.32
Red cypress	Posts	4 pieces, 9″×9″×13′	351	8.00	2.81
White oak	Sills	1 piece, 9″×10″×15′	112	11.00	1.23
Longleaf pine	S. braces	1 piece, 7″×7″×18′ 6″	76	8.00	.61
			1,641		16.28
		Cost of iron			3.17
		Total cost of panel			19.45

IRON.

Size.	Number of pounds.	Cost per pound.	Total cost.
		Cents.	
Bolts (stringers), 8 pieces, ¾″×38½″, at 6.15	49.2	4	$1.97
Drift bolts, 2 pieces, 1″×23″, at 7.5	15	4	.60
Washers, O. G., 16 pieces, 3″ diameter, at 1.25	24	2½	.60
Total			3.17

STRINGERS.

The stringer is the first thing to be considered in the design of timber trestles. It is the most important member of the structure and the most difficult to design economically. The cost of a stringer is a function of not only the number of feet, B. M., contained therein, but also of the height or maximum cross-sectional dimension.

The Atlanta Lumber Company, of Atlanta, Ga., quote the following relative prices for longleaf-pine timber such as would be used for stringers: For sizes 12 inches or under, $8 per 1,000 feet, B. M. For sizes over 12 inches add $1 per inch per 1,000 feet, B. M.

Assuming that these prices represent the average conditions, fig. 5 shows the relative cost of stringers of different heights but of equal cross-bending strength and stiffness.

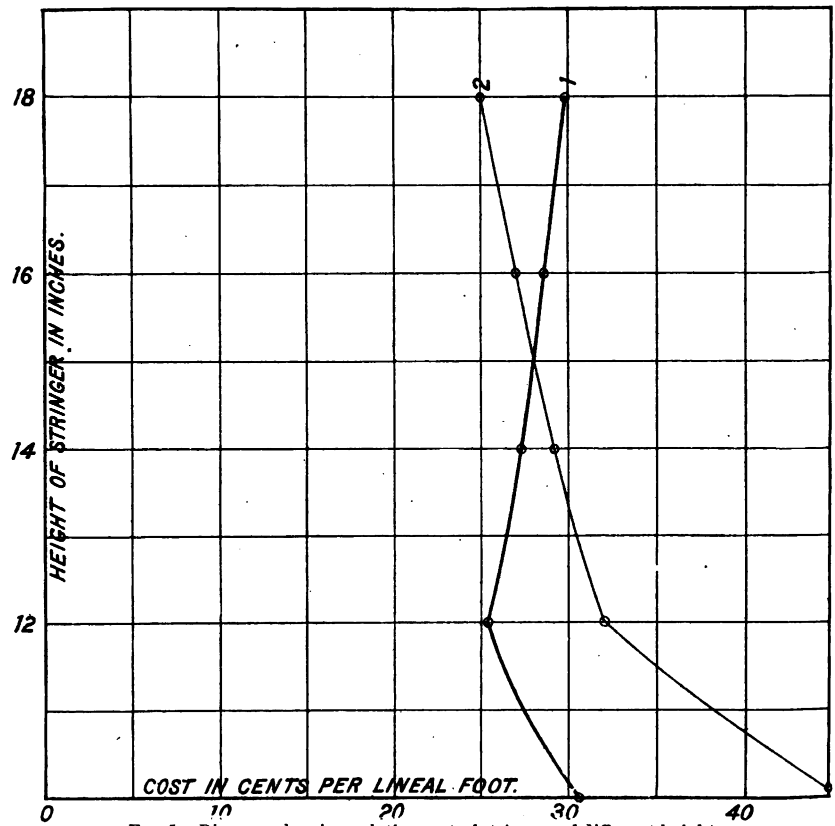


FIG. 5.—Diagram showing relative cost of stringers of different heights.

The heavy line shows the relative cost of stringers of different heights, but designed to develop the same safe unit stress in the extreme fiber, while the light line shows their cost when designed to develop the same maximum allowable deflection of one two-hundredth of the span.

COST OF STRINGERS OF EQUAL STRENGTH.

For uniform load in cross bending:

$$R^* = \frac{3\,W\,l}{4\,b\,h^2}$$

$$\text{or } b = \frac{3\,W\,l}{4\,R\,h^2} \text{ from which } b\,h = A = \frac{3\,W\,l}{4\,R\,h}$$

*See Appendix 1.

where A=area of cross section in square inches, and $\frac{A}{12}=\frac{W\,l}{16\,R\,h}$=number of feet, B. M., per linear foot of stringer.

Let c equal the cost per foot, B. M., for height, h; then

$$\frac{Ac}{12}=\frac{W\,l\,c}{16\,R\,h} \tag{1}$$

equals the cost per linear foot of this height of stringer.

For any other height we have, as the cost per linear foot of stringer of equal strength

$$\frac{A'c'}{12}=\frac{W\,l\,c'}{16\,R\,h'} \tag{2}$$

Dividing (1) by (2) we have

$$\frac{\frac{Ac}{12}}{\frac{A'c'}{12}}=\frac{c\,h'}{c'\,h} \tag{3}$$

For example, the cost per linear foot of a 12-inch stringer is to the cost per linear foot of a 14-inch stringer of equal strength as 8 times 14 is to 10 times 12, equal to $\frac{14}{15}$.

Having, therefore, computed the cost per linear foot for the given conditions of span and load by equation (1), the cost for other heights may be found by equation (3).

COST OF STRINGERS OF EQUAL STIFFNESS.

The modulus of elasticity *

$$E=\frac{5\,W\,l^3}{32\,\Delta\,b\,h^3}$$

for uniformly distributed load.

W = total load on stringer in pounds.
b = breadth in inches.
h = height in inches.
l = length of stringer in inches.
Δ = deflection at middle in inches.

Then $b=\frac{5\,W\,l^3}{32\,E\,\Delta\,h^3}$ or since $\Delta=\frac{l}{200}$ $\quad b=\frac{1{,}000\,W\,l^2}{32\,E\,h^3}$

or

$$b\,h=A=\frac{1{,}000\,W\,l^2}{32\,E\,h^2}$$

* See Appendix 1.

The Atlanta Lumber Company, of Atlanta, Ga., quote the following relative prices for longleaf-pine timber such as would be used for stringers: For sizes 12 inches or under, $8 per 1,000 feet, B. M. For sizes over 12 inches add $1 per inch per 1,000 feet, B. M.

Assuming that these prices represent the average conditions, fig. 5 shows the relative cost of stringers of different heights but of equal cross-bending strength and stiffness.

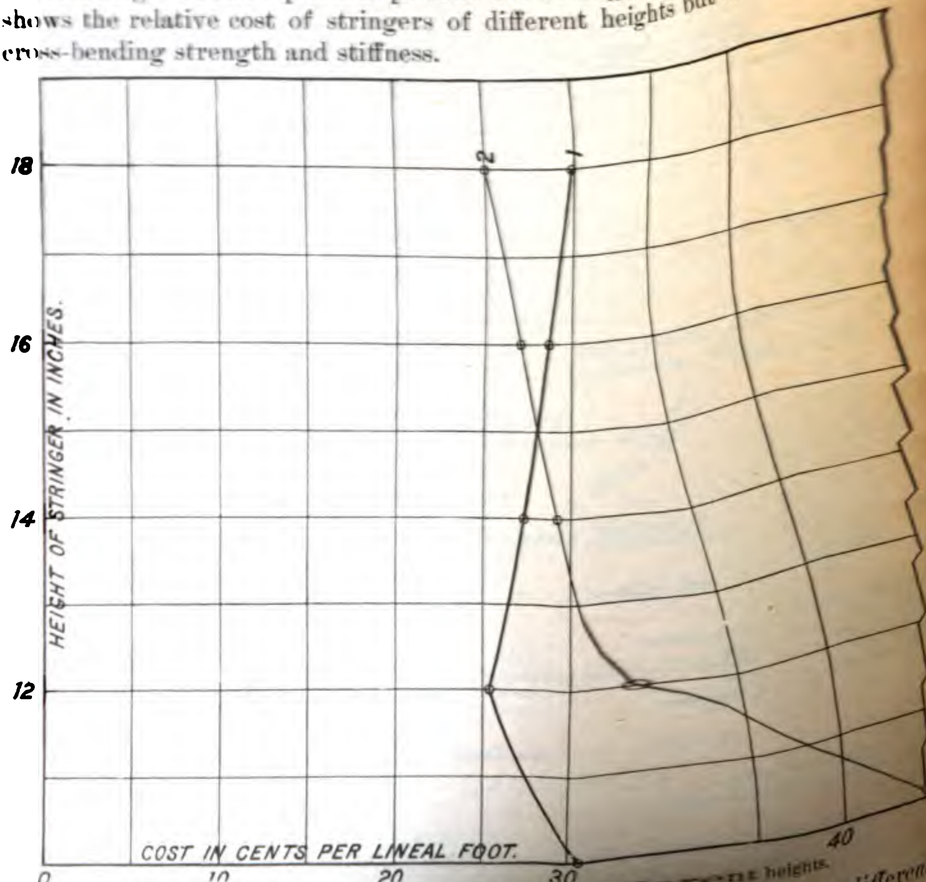


FIG. 5.—Diagram showing relative cost of stringers of different heights.

The heavy line shows the relative cost of stringers of different heights, but designed to develop the same safe unit stress in the extreme fiber, while the light line shows their cost ... develop the same maximum allowable deflecti... of the span.

COST OF STRINGE...

For uniform load in cross be...

or $h = \frac{3W}{4E}$...

where A=area of

ber of feet. B. M.

Let c equal the

equals the cost per

For any other

of equal strength

Dividing

For example,

cost per linear foot of

is to 10 times

Having, therefore,

conditions of

may be found by equation

number

The modulus of

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ever, much better structure,

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more force than the assertion that a

is not so strong as one containing but ten.

not exceeding 3 or 4 feet in length, is of

tion of a well-designed stringer at the end of

exceed a quarter of an inch for the pro

or, as before,

$$\frac{Ac}{12} = \frac{125 \, W \, l^2 \, c}{48 \, E \, h^2} \tag{4}$$

equals the cost per linear foot of stringer of height h.

For any other height we have

$$\frac{A_1 c_1}{12} = \frac{125 \, W \, l^2 \, c^1}{48 \, E \, h_1^2} \tag{5}$$

Dividing (4) by (5)

$$\frac{\frac{Ac}{12}}{\frac{A_1 c_1}{12}} = \frac{c \, h_1^2}{c_1 h^2} \tag{6}$$

That is to say, the cost per linear foot of stringers designed to give a maximum deflection of one two-hundredths of the span decreases as the square of the height, and increases with the first power only of the cost per foot, B. M. The cost, therefore, will be a decreasing quantity according to the present method of fixing the prices.

These two curves necessarily intersect. The point of intersection is a very important point and may be found for the general conditions of species and length of span by solving equations (1) and (4) as simultaneous equations thus:

$$\frac{W l c}{16 \, R h} = \frac{125 \, W l^2 \, c}{48 \, E \, h^2}$$

from which we get

$$\frac{h}{l} = \frac{2{,}000 \, R}{48 \, E} \tag{7}$$

For the case under consideration the span was 14 feet and the material longleaf pine, for which (see Table IV) R = 1,550 and E = 720,000. Hence, $\frac{h}{l}$ for this case is equal to $\frac{1}{11.1}$ and $h = \frac{168}{11.1} = 15.1''$, as shown in the diagram.

For heights below that of the point of intersection stringers should be designed by the deflection formula; for greater heights, by the cross-breaking formula. The most economical height is that at the point of intersection and for longleaf-pine stringers is $\frac{1}{11.1}$ of the span.

Assuming a height of 14.5 inches for stringers, the necessary width under each rail is equal to 21.8 inches, making three pieces 7¼ by 14½ inches by 14 feet long. From Table VII we see that the length of end bearing or grip on the cap necessary for these stringers is $6'' + 30$ per cent $= 6'' \times 1.3 = 7.8''$. To provide for weathering the stringers are increased one-half inch on each cross-sectional dimension, making three pieces under each rail 7¾ by 15 inches by 14 feet long. Now, since 7.8 inches grip are required for the grip on the cap, either a corbel must be used or the stringers given a full bearing on the cap.

CORBELS.

A corbel is deemed best for this case, as it has several advantages, among which are the following:

(1) It not only supports but unites the abutting stringers, forming a portion of the bond between them.

(2) It stiffens the joint, materially decreasing the strain in the stringers.

(3) When employed single-span lengths of stringers may be used, these being cheaper and more readily furnished than the double-span length. The double-span length is of course better.

(4) Large beams, which stringers usually are, are particularly liable to shear along their neutral axis. They will fail in this manner at less than half the shearing strength per square inch, as indicated on a small test. The reasons for this are: (1) That a large beam is apt to contain a portion of the heart center of a log, which is likely to be ring shaken; (2) with old trees—and the trees from which such sizes are cut are usually old—the heart center is much below the average quality of the cross section; (3) large pieces are particularly liable to check in seasoning. Now, the bolt through the corbel and stringer does excellent service in increasing the resistance to failure by shearing. In fact, even when corbels are not used, a bolt through the ends of the stringers would be a wise precaution. It would be necessary, however, to tighten these occasionally until the timber had thoroughly seasoned.

(5) Corbels in many cases furnish the only means of obtaining sufficient bearing value for the stringers.

Many companies will not use corbels, claiming—

(1) That they increase the cost in labor, lumber, and iron.

(2) That they increase the number of joints and hence the number of places for the beginning of decay.

(3) That they do not, after all, increase the bearing area of the stringer, since, as the latter deflects, the whole load is brought upon the ends of the corbel.

With regard to the first item, by comparing the cost of designs shown on figs. 2 and 3, we see that the corbel design costs in material 68 cents more per bent. The additional cost in labor would be very nearly offset, probably, by the greater facility in handling smaller and fewer pieces. Perhaps 75 cents would be a fair estimate of the extra cost of corbel construction. It gives, however, much better structure, and would undoubtedly secure enough additional length of life to more than pay for the extra original cost.

The second objection is of no more force than the assertion that a chain containing eleven links is not so strong as one containing but ten.

The third claim, in a corbel not exceeding 3 or 4 feet in length, is of no moment. The deflection of a well-designed stringer at the end of a 4-foot corbel will not exceed a quarter of an inch for the proof load.

Even if the stringer crushed all this amount, it would still be able to crush as much more before being in danger. This would not happen, however. The corbels may be beveled toward the ends an eighth of an inch if desired, but it is scarcely necessary.

In the last design, by using four stringers 7¼ by 14½ inches each, with a 12-inch cap, the corbels are avoided, sufficient bearing area being obtained without them.

POSTS AND CAPS.

The posts in these last two designs are much lighter than those in the first, but, before weathering begins, have a factor of safety of about 9. Taking off one-half inch from the cross-sectional dimension to allow for weathering, after being in service for some years they still have a factor of safety of 7½. The dimensions were obtained as follows:

Assuming four posts in the bent, we have $\frac{91,800}{4}$ (22,950 pounds), to be carried by each post, or 11.48 tons. These posts will be about 11 feet 8 inches long and have a ratio of $\frac{l}{d}$ = about 20, probably. From an inspection of Table IV we find that for this ratio a 7 by 7 inch longleaf-pine post will suffice.

With an oak cap, the safe bearing value of which is 400 pounds per square inch, we will require an area of cap and sill at the end of each post of $\frac{22,950}{400} = 57.4$ square inches. Adding to this the mortise area, we have 72.4 square inches which will require a post a trifle over 8.5 inches square. Then adding a half inch to each dimension to allow for weathering, we have a 9 by 9 inch post, requiring caps and sills of the same size. But with posts of this size we may use cypress or any of the weak, cheap, but durable timbers.

The designs in figs. 3 and 4, though capable of carrying twice as much load as that shown in fig. 2, show a saving of $5 per span, equal to 36 cents per linear foot of track, and 28 per cent less timber.

Assuming that this would be representative of one-half the total mileage of timber trestle bridges, i. e., 1,000 miles, we have a total saving every nine years of $1,900,000, which is equal to an annual expenditure of $211,000. This capitalized at 4 per cent gives a capital of $5,275,000. These 1,000 miles of trestle use annually about 120,000,000 feet, B. M., of valuable timber, 35,000,000 feet of which might readily be saved.

The tables of cost accompanying these designs upon which the above figures have been based are, of course, subject to great modification, depending upon the location, condition of the market, etc. It is thought, however, that they give a fair representation of the average conditions.

APPENDIX I.

EXPLANATION OF FACTORS OF STRENGTH.

Cross breaking.—From the cross-breaking test are obtained the modulus of strength at rupture (R), the modulus of elasticity (E), and the elastic resilience per cubic inch (r).

The modulus of strength at rupture is the intensity (in pounds per square inch) of the stress upon the extreme fibers of a beam at the point where, and at the time when rupture begins.

For example, take a longleaf-pine beam of any length and height and 10 inches wide, loaded to the point of failure. The value of the modulus of rupture for this species is 5 by 1,550 (see Table IV) = 7,650 pounds per square inch. Now, if we conceive a layer of extreme fiber a tenth of an inch in thickness and running the full width of the beam, then the actual load on this square inch of material, tending to pull the fibers apart or crush them together—depending upon whether this layer was taken from the convex or concave side of the beam—is 7,650 pounds.

The stress on the extreme fiber (f) is a function of the size of the load, the method of loading, and shape of the piece.

For rectangular beams of uniform cross section

$$f=\frac{3\,W\,l}{4\,b\,h^2}, \text{ for beam uniformly loaded,}$$

$$\text{and } f=\frac{3\,W\,l}{2\,b\,h^2}, \text{ for concentrated load in middle;}$$

where

W = total load on beam in pounds,
l = length of beam in inches,
b = breadth of beam in inches,
h = height of beam in inches,

the latter dimension being measured parallel to the direction of the load.

If W is the proof load on the beam, then f becomes equal to R, the modulus of strength at rupture.

The modulus of elasticity is the ratio of

$$\frac{\text{Unit stress (in pounds per square inch)}}{\text{Unit distortion (expressed as fractional part of length)}}$$

Thus, if 30,000 pounds will stretch a bar of iron 1 inch square one one-thousandth of its length, the modulus of elasticity for that iron is

$30{,}000 \times 1{,}000 = 30{,}000{,}000$ pounds per square inch. Or, in cross bending, if a bar be bent so that the stress per square inch on the extreme fiber directly under the load is 30,000 pounds, and the elongation of this fiber in a length of 1 inch be one one-thousandth of an inch, then will the modulus of elasticity be again 30,000,000 pounds per square inch. For the different species of wood this factor varies from about 300,000 to 3,000,000. It is independent of the size and shape of the piece, as also of the method of loading.

For uniformly loaded beam,

$$E = \frac{5 f l^2}{24 \Delta h} = \frac{5 W l^3}{32 \Delta b h}$$

and for beam with concentrated load in the middle,

$$E = \frac{f l^2}{6 \Delta h} = \frac{W l^3}{4 \Delta b h^3}$$

where f is stress on the extreme fiber for the given condition of loading and Δ is the deflection in inches, the other quantities being as before.

The elastic resilience of a beam is the product of one-half the load into the deflection at the loaded point. For a beam loaded with numerous concentrated loads the resilience is one-half the total sum of each load into the deflection of beam at that point. This quantity is a function of the volume of the beam.

The elastic resilience per cubic inch, as given in Table IV, is the above quantity for rectangular beams, divided by the volume. It is a measure of the amount of shock that may be absorbed, without injury, by rectangular wooden beams loaded in the middle.

The other factors of strength are very simple and do not require explanation.*

* For the method of making the experiments, etc., see Bulletin 8, pp. 4–8.

APPENDIX II.

REVIEW OF THE FOREGOING PAPER BY MR. G. LINDENTHAL,* CHIEF ENGINEER OF THE NORTH RIVER BRIDGE COMPANY.

The designing of trestle structures has only in recent years been undertaken by engineers; formerly it was left entirely to the practical judgment of bridge carpenters, aided by a few general rules as to strength and section of timbers issued to them by railroad managers. Usually the sizes of timber most readily handled were employed, and variation of section was avoided, as likely to lead to delay in getting the material from the sawmills. Timber being cheap and labor dear have sometimes led to constructions far from economical in an engineering sense.

Trestle bridges on almost all railroads are regarded as temporary structures, to be replaced with stone or iron structures and with filling. If the financial condition of the railroad does not permit the permanent work to be done before the timber structure becomes unsafe from wear or rottenness, it must be replaced with a new timber structure, as being the cheaper; yet it never loses its temporary character. It is true, however, that much money can be saved in the correct designing of trestles.

The proposition to classify timber structures according to the moisture they contain and to use different values for dimensioning for different moisture classifications would, I think, unnecessarily complicate the designing. It is better and simpler, as well as safer, to assume that all timber going into trestles is green and that only the unit stresses for green timber should be used in dimensioning it.† According to location and use, the timber will either remain green or will get seasoned; in the latter case there is no harm done—the timber simply gets stronger. Simplicity will always remain a valuable rule in designing such structures.

Insistence on selecting good timber, free of knots and wind shakes, is justified, and the inspection of the timber in that regard should be very rigid.

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*As explained in the letter of transmittal, the paper by Mr. A. L. Johnson, C. E., was submitted to two leading bridge engineers for review, and extracts from the expression of their views thus elicited are appended, believing that they will add to the value of Mr. Johnson's paper and to its appreciation by the public.—B. E. F.

†This is practically what the author of the paper has assumed. The moisture for all timber in trestles has been taken at 18 per cent, which is called "half-dry." The increase in strength from the green to this condition is so slight as to be immaterial.

Regarding the details of construction, it is quite proper to call attention to the stupid disproportion in strength of the columns, caps, and sills, and particularly to the inadequate bearing areas of stringers upon caps and of columns against caps and sills.

The use of mortises for joining the columns to the caps and sills, however, should be discouraged. It reduces the bearing areas, increases the amount of work in the fitting, provides places in the lumber for the accumulation of moisture, and is in every respect an unsuitable construction, borrowed from ancient roof building, where it may do less harm, being all the time under cover and absolutely dry.

I inclose a sketch, showing a much cheaper and better construction (fig. 6). In place of two vertical and two slanting legs, use four vertical legs. It is obvious that the fit of the outside slanting legs requires accurate work, consequently takes more time and money, whereas the four vertical legs being of the same length, require no special fitting.

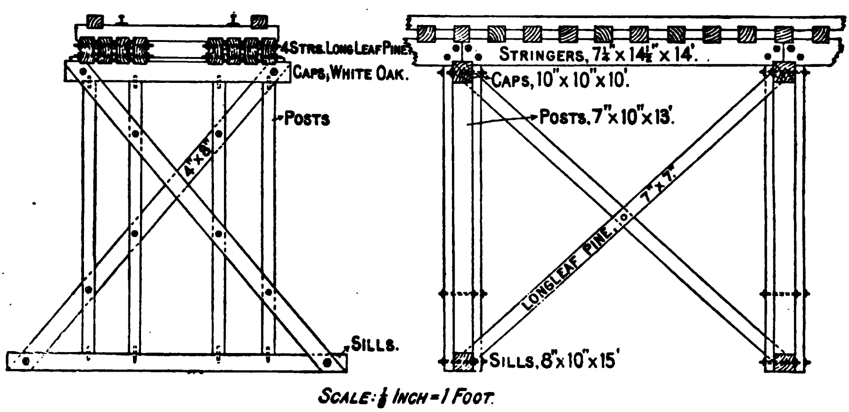


FIG. 6.—Showing construction with four vertical legs.

Next, use three-quarter-inch iron dowel pins to connect columns to caps and sills, and further use diagonal braces bolted through with three-quarter-inch bolts and large 2-inch washers, so that the trestle bent will be a compact, rigid structure which, if necessary, can be lifted with a hoist and set up in place.

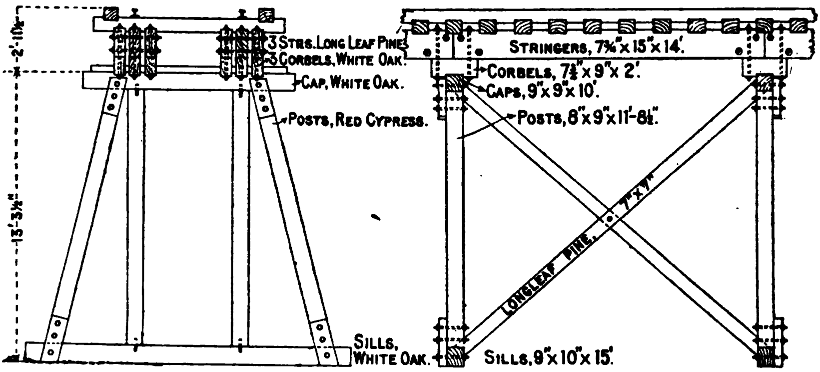
I inclose another sketch, showing two vertical legs and two slanting, outside legs, using dowel pins, wooden splice pieces, and bolts for connecting the whole in the one rigid frame (fig. 7).

By using dowel pins the bearing of post on caps and sills is not impaired. The posts can be proportioned without making additions for mortises. The work of fitting is very much reduced. The larger amount of ironwork in the form of bolts and washers is *not* a disadvantage, since these can be used again in the renewal of the structure.

The detail of corbel bearings, as provided in Mr. Johnson's design, is to be commended; but I would not commend using short stringers the length of only one panel. The stringers should always be the length

of two panels, with alternating joints in the two, three, or four stringers under each rail, as may be found necessary.

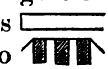
One point to be watched in trestle construction is to prevent the lodgment of cinders in the structure, which would set it on fire. The



SCALE: ½ INCH = 1 FOOT.

FIG. 7.—Showing construction with two vertical and two standing legs.

cinders will usually lodge between the ties on top of the stringers or between the stringers on top of the cap. For this reason it is a good practice to cover the top of the stringers, from end to end, with a galvanized-iron sheet bent down over the edges. It will not only protect the stringers against the weather, but it will also largely protect the structure against fire, on account of which a large amount of wooden trestlework each year has to be renewed. Another method used with success against fire is planking over the stringers and gravel between the ties, used by Mr. Bouscaren on a long trestle near New Orleans.



As regards longitudinal bracings between trestle bents, no general rules would be applicable. It will always depend upon the local conditions. Where the trestle is high or on a curve or on uncertain bottom, special provisions of bracings will have to be made.

It may not be amiss to say—although not directly connected with the subject under discussion—that no better way of economizing timber in the construction of railroads could be devised than doing away with trestles altogether.

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APPENDIX III.

NOTES BY MR. WALTER G. BERG, PRINCIPAL ASSISTANT ENGINEER OF THE LEHIGH VALLEY RAILROAD.

The first section of the paper contains pertinent criticisms of the existing practice, and presents valuable suggestions and exceedingly interesting tables. The demonstration of the necessity for a more regular and uniform theoretical practice, based upon scientific research, in place of the chaotic conditions, as exhibited by Table III, is certainly apparent after a perusal of this part of the paper.

The most valuable information for the profession at large is contained in Table IV, which represents the data obtained to date from the very extensive series of timber tests of the United States Forestry Division, supplemented by the best information available in regard to other authenticated tests. This table can be considered in the nature of an advance sheet, giving the grand averages of the results of the tests mentioned up to date, and deserves to be well circulated and regarded as the latest authentic information till supplemented by future publications of the Forestry Division.

The fact, mentioned by the author of the paper, that structural iron and steel is invariably tested with the greatest degree of care, while timber for structural work is seldom given more than a cursory eye inspection for visible defects, is traceable to the fact that very little reliable data have thus far been available on which to base the strength of timber. The great variations existing in the coefficients of strength for the different parts of a timber trestle, as assumed in practice, brought out so clearly in Tables II and III, is a silent but potent argument in favor of timber tests on a large scale, as being conducted by the United States Forestry Division. These investigations, extended to include all the principal varieties of timber in use, and especially tests of large size timber struts, will have a greater tendency to effect a decided economy in the use of timber throughout the country than the presentation of individual views as to how to improve on existing practice in particular cases. With reliable knowledge of the proper coefficients of strength that can be allowed for the various parts of a timber structure, it will be perfectly safe and consistent to expect that the parties interested in the economical construction of a structure will be only too glad to avail themselves of any reasonable economy that can be effected without assuming any undue risks.

Table III illustrates, in general, that the railroad practice of the day is working with approximately correct units of strength as far as the

cross breaking of stringers is concerned, but that the proper units for shearing and for bearing values across grain are either not known or entirely disregarded in a great many cases. Similarly the posts are apparently designed out of reason, but it will be noted that they are invariably too heavy, which fact is traceable in part to certain practical reasons to be mentioned later.

The statement of the author of the paper that "although the stringers in cross breaking have a factor of safety of 5, and the posts have a factor of safety of 20, the structure as a whole has a factor of safety of only 2 approximately" is not warranted as to the conclusion reached, with the whole meaning apparently thereby conveyed. The meaning would seem to be that some vital part of the structure has a factor of safety of 2 only, and hence the structure would be dangerous if loaded to twice the assumed load. As mentioned above, the low factors of safety, according to Table III, are invariably for bearing values. Timber may indent badly, due to an excessive bearing strain across grain, but this would not necessarily prove disastrous. It is more a serious question of maintenance expenses than of absolute danger to the structure.

It is hardly correct, therefore, to conclude, owing to the small bearing surfaces at the ends of stringers or of the caps on the posts, that the prevailing practice of timber-trestle construction is subject to as low a factor of safety as 2 in one of its principal parts absolutely affecting the safety of the structure.

As mentioned above, Table IV contains the most valuable information to the profession at large presented in the paper. It gives the results of the very valuable United States Government tests, mentioned above, corrected, adjusted, and supplemented by the author of the paper, whose professional standing and work in connection with these Government tests qualify him to present this information in the most authentic manner. The profession owes thanks to Mr. Johnson for compiling this table with the accompanying explanatory remarks. It would have, perhaps, made the table more valuable to have given extreme breaking unit stresses in place of safe unit stresses, leaving it to each individual designer to select his own factors of safety, while a general recommendation as to the desirable factors of safety could have been added.*

Table V is also of great practical benefit, as it is based on the latest and most reliable coefficients for the cross-breaking stress of timber.

Table VI would be of even more value to the profession than Table V, provided more information were presented relative to the new formula for columns presented in the paper upon which the table is based. It would be desirable to know the actual number and kind of tests from the results of which the new formula was developed. Presumably the information will be supplied in a future publication of the Forestry

* They can be obtained by multiplying the safe values by the factors of safety given.

Division. A formula for large timber columns, the correctness of which can not be questioned, is a great desideratum, and would be a boon to engineers, architects, and in fact all designers of structural work.*

Table VII is indicative of the importance attributed by the author of the paper to the proper proportioning of timber structures for bearing of timber on timber across grain. The point at which an indentation due to the bearing of timber on timber across grain becomes dangerous or objectionable depends upon individual views and local conditions. The units for crushing strength across the grain are assumed by the author of the paper as being the stresses at which an indentation of 3 per cent of the height of the stick would take place.

In regard to inspection of timber, the author of the paper outlines the principal points affecting the strength of timber, which points form the basis of the investigations and tests now being conducted by the Forestry Division. There is no doubt that, with additional systematic and reliable information furnished on these different questions, the proper inspection of timber will be greatly assisted and benefited, but, as the author of the paper states, the present time is premature for the formulation of rules for scientific inspection of timber.

Referring now to the second division of the paper, in which the present practice of trestle construction is compared with designs proposed by the author of the paper, it should be said, in a general way, that it is to be regretted that, in selecting a structure from everyday practice to illustrate the actual practical economies that can be introduced, owing to our present better information on the subject of strength of timber, the author did not select some other timber structure in place of a trestle. The different parts of a railroad trestle can not always be designed so as to be subject to or to correspond to mathematical calculations, as there are certain conditions in the actual construction or erection work and subsequent use of the structure which predominate in establishing the design and the dimensions far more than theoretical calculations of the strength of individual parts.

The principal features of the proposed new designs, as compared with the present practice, consist in the recommendation for the introduction of corbels, in which case the caps are reduced to 9 inches; the scaling down of the posts to correspond to the new formula for wooden columns; the proper designing of stringers, not only to be safe for cross breaking and deflection under load, but especially for bearing value at end of stringer, and a corresponding general scaling down of all other parts, such as sills, braces, etc., although no attempt is made to warrant these last reductions of sizes by calculations.†

* The data for the development of such a formula are being collected, but are not yet sufficient to warrant conclusions.—B. E. F.

† Not true as far as caps and sills are concerned. (See p. 28.) The braces have not been computed in the paper; in fact, they are not mentioned. There is a radical change in them, however, and I computed them before putting them in.

On the design in fig. 2, present practice, it will be noticed that there is one brace in each panel, while in the new designs there are two braces in every other panel.

The writer agrees with the author of the paper that the bearing surfaces at the ends of stringers and of caps in railroad trestles are usually too small, and that in consequence indentations of the timber take place, which means a destroying of the fiber and, in connection with moisture and constant working, subsequent speedy decay. But it will be impossible to follow the author of the paper to the extent of recommending the introduction of corbels under the ends of stringers. This may be due in part to the individual objections that the writer has always had against the use of corbels. These objections have not been overcome by a perusal of the points presented by the author of the paper in favor of corbels, and an examination of the design recommended, showing corbels, fails to reveal wherein the bearing surface on top of the cap is improved by the introduction of a corbel under each stringer, considering that the corbels as proposed are of the exact width as the stringers. The author of the paper must either assume that the corbels reduce the clear span or else that only the panel on one side of the bent is loaded. Both assumptions are wrong from a practical standpoint. As above stated, it is impossible for the writer to see how a 7¾-inch corbel resting on a 9-inch cap, giving about 70 square inches of bearing surface on top of the cap, can be an improvement over a stringer of the same width as the corbel, resting on a 12-inch cap, giving 93 square inches of bearing surface, or, according to the "example of present practice," over an 8¼-inch stringer resting on a 12-inch cap, giving 99 square inches of section.

The introduction of corbels is objectionable on account of the extra cost, without any commensurate improvement of the design, and owing to the undesirable feature of bringing more timber against timber, especially on horizontal and boxed surfaces, increasing the liability of decay. The recommendation (?) of the author of the paper to use short stringers in place of double-length stringers is a disadvantage; while a screw bolt through the corbel and stringer, tending to increase the resistance of the beam to failure by shearing, would prove, if introduced solely for this purpose, a foolish precaution in place of a wise one, for the reason that with the seasoning of the timber the grip of the bolt would be lost unless kept regularly tightened up. To do this to such an extent as to be of practical benefit would be impracticable. There are cases where corbels laid flat, i. e., made wider than the stringers, will increase the bearing value on the cap. There are also cases where the elevation of the rails on curves on trestlework can be best accomplished by the introduction of corbels, giving an opportunity to vary the elevation according to the thickness of the corbel. But as a steady,

This latter arrangement allows the braces to be attached at their intersection by means of a bolt. Hence, instead of computing a strut large enough to transmit the thrust along the full length of the diagonal, about 18 feet, we need only determine the area necessary to transmit it half that distance. That is, in the first case we have a column 18 feet long, and in the latter case columns only 9 feet long, so that they do not need to be so large.—A. L. Johnson.

everyday practice, corbels can not be considered as desirable, and as designed and recommended by the author of the paper there is a decrease in bearing value on the cap in place of an increase.*

In relation to the posts, the aim of the author of the paper would be evidently to scale down the posts of the different bents to such odd sizes as would correspond with the mathematical calculations of strength. For the example presented of a post about 12 feet high, the author recommends a 9 by 9 inch stick in place of a 12 by 12 inch stick, or, in other words, the actual size in use is about 75 per cent larger than the theoretical requirements. This might seem to indicate that a similar disproportion prevails for all lengths of posts. Attention should be called, however, to the fact that even theoreticians would hardly desire to call for a post much less than 9 by 9 inches on a trestle for supporting heavy locomotives, even for very short posts, while the longer the post gets the more nearly the theoretical size will correspond to current practice, which is to use practically 12 by 12 inch sticks throughout. The comparatively low height of trestle assumed by the author of the paper for demonstrating the case does not really represent a fair average of the trestles in the country as far as the proportion of theoretical to actual sizes of posts is concerned. In other words, the grand average of actual current work would show that the timber used for posts is closer to the theoretical requirements than in the example assumed for illustration.

The use of 12 by 12 inch stuff indiscriminately, while wasteful of timber in certain cases, is warranted by a great many practical features which absolutely control the situation and predominate far above theo-

* The reviewer here and elsewhere has missed the main point made for the use of corbels as proposed, namely, that they as well as caps and sills be of oak or some similar hard wood, so that a smaller bearing area will yield twice the strength. The bearing areas have been made sufficient to give a factor of safety of 3 for oak. The corbels were used to give the stringers sufficient bearing area.—A. L. J.

To this rejoinder Mr. Berg had an opportunity of replying, the reply being as follows:

"If corbels and caps of a trestle are made of a timber like oak, with a relatively high unit resistance to crushing across grain, as suggested by Mr. Johnson, then the deficient bearing at the ends of the usual stringers in practice will be obviated. It is a question, however, whether in practice it would be feasible to make such a distinction and to utilize several species of timber. For special work the proposition is all right, but for the general run of railroad work it would be difficult to introduce this innovation, especially in sections of the country where hard timber for the corbels and caps would be more costly or difficult to obtain. Practical men also claim (although how correctly I am unable to say) that different species of timber in contact with each other will rot quicker than if only one kind of timber is used, and this statement has especially been made frequently in connection with bringing oak timber and soft pine timber in contact with each other."—W. G. B.

[The last objection has no physiological basis. The former illustrates the penny-wise pound-foolish policy which unfortunately prevails with many if not most railroad companies, to the detriment of the public and the stockholders, and the necessity for competent demonstration of the financial superiority of stable, lasting structures on permanent roads.—B. E. F.]

retical calculations. Space is here too brief to enumerate all possible reasons for this apparently wanton use of 12 by 12 inch lumber, but the mention of a few will suffice to indicate the general trend of the practical questions which the writer has in mind. A railroad corporation has to keep a large stock of lumber on hand for emergencies, sudden calls for new work not allowing time to obtain lumber from the mills, and for other reasons. The uncertainty as to where the timber is to be eventually used makes it impossible to have the timber cut to specific sizes, and, again, to attempt to run a large lot of sizes would be more wasteful in the end than to maintain a few stock sizes only. Lumber can be bought more cheaply by giving a general order for "the run of the mill for the season" or "a cargo lot," specifying approximate percentages of standard stringer size, of 12 by 12 inch stuff, 10 by 10 inch stuff, etc., and a liberal proportion of 3 or 4 inch plank, all lengths thrown in. The 12 by 12 inch stuff, etc., is ordered all lengths, from a certain specified length up. In case of a wreck, washout, burn out, or sudden call for a trestle to be completed in a stated time, it is much more economical and practical to order a certain number of carloads of "trestle stuff" to the ground and there to select piece after piece as fast as needed, dependent only upon the length of stick required. When there is time to make the necessary surveys of the ground and calculations of strength and to wait for a special bill of timber to be cut and delivered, the use of different sizes for posts in a structure would be warranted to a certain extent.

The reasons presented indicate, therefore, that while it is not true economy as far as the general timber consumption of the country is concerned to use 12 by 12 inch sticks indiscriminately for posts, caps, and sills of trestles, still, in the majority of cases, it is true practical economy as far as the interest of the party who is paying for the work is concerned.

Another very important feature to consider in the designing of trestles is not only to give a proper factor of safety as established from timber tests, but to design those parts the failure of which would prove absolutely disastrous in such a way as to give an additional guaranty of safety against the racking of the structure from the unknown strains and peculiar conditions resulting from the impact of fast-moving trains with heavy concentrated wheel loads passing over the trestle, the centrifugal force caused by high-speed trains upon curved trestles, the lateral swaying induced by wind or the wobbling of the train back and forth within the play of the gauge even on a straight track, and other similar features.*

There are also other practical points to consider, such as, for instance, the possible shifting of the bearing caused by the undermining of some mudsills, or the loosening or breaking of one of the posts, in which

*The factor of safety of 5 was an average value reported by the engineers of the railroad companies and is by the author considered sufficient only if applied to the "minima" values of strength.—A. L. J.

case a greater strain than originally contemplated is thrown on another part of the trestle. In some trestles the batter posts are spaced so far from the main posts that the main posts have to be considered as practically carrying all the load, and no one would desire to trust a 9 by 9 inch stick in that case. If the stringer is three-ply, or especially four-ply, as shown by the author of the paper in one of the recommended designs, the strain on all the stringers would not be perfectly equal and the bearing on the cap would not fall over the post in every case, which would produce certain cross strains in the cap, requiring consideration.* In addition, the fact that the cap receives a terrible punishment in the shape of direct blows from the passing rolling stock has caused actual experience to demonstrate that frequently even 12-inch sticks, especially when made of brittle timber,† will not stand, but break off.

Another practical reason for the employing of larger sizes than theoretically required in certain parts of a trestle is that certain sticks, even with the best inspection and light on the subject that we now have, will rot before others, or certain bearings will become defective long prior to others. The very best bridge inspection and supervision can not pretend to discover and repair immediately every defect as soon as it appears. Therefore an excess of strength is required to allow for the premature decaying of certain parts until such time as repairs to the structure are feasible.

In the writer's opinion, the proper method to pursue to cause true economy in the designing of trestle bridges is to circulate and spread widely the knowledge of the results obtained from the very valuable series of timber tests made by the Government, which data should be carefully studied by the designers of structures. The more economical use of timber, with due regard to the true safety of the structure, will surely and gradually follow in direct ratio as the number, reliability, and value of the Government tests increase.

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*But the responsibility is divided, should any one prove defective.—A. L. J.

†The oak cap will stand better.—A. L. J.

APPENDIX IV.

REPORT OF A COMMITTEE OF THE AMERICAN INTERNATIONAL ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS ON "STRENGTH OF BRIDGE AND TRESTLE TIMBERS."*

Your committee appointed to report on "Strength of bridge and trestle timbers, with special reference to Southern yellow pine, white pine, fir, and oak," desire to present herewith, as part of their report, the very valuable data, compiled by the chairman of the committee, relative to tests of the principal American bridge and trestle timbers and the recommendations of the leading authorities on the subject of strength of timber during the last twenty-five years, embodied in the appendix to this report and tabulated for easy reference in the accompanying tables (I to IV).

The uncertainty of our knowledge relative to the strength of timber is clearly demonstrated after a perusal of this information, and emphasizes, better than long dissertations on the subject, the necessity for more extensive, thorough, and reliable series of tests, conducted on a truly scientific basis, approximating as nearly as possible actual conditions encountered in practice.

The wide range of values recommended by the various recognized authorities is to be regretted, especially so when undue influence has been attributed by them in their deductions to isolated tests of small-size specimens, not only limited in number, but especially defective in not having noted and recorded properly the exact species of each specimen tested, its origin, condition, quality, degree of seasoning, method of testing, etc.

The fact has been proved beyond dispute that small-size specimen tests give much larger average results than full-size tests, owing to the greater freedom of small selected test pieces from blemishes and imperfections and their being, as a rule, comparatively drier and better seasoned than full-sized sticks. The exact increase, as shown by tests and by statements of different authorities, is from 10 to over 100 per cent.

Great credit is due to such investigators and experimenters as Profs. G. Lanza, J. B. Johnson, H. T. Bovey, C. B. Wing, and Messrs. Onward Bates, W. H. Finley, C. B. Talbot, and others, for their experimental work and agitation in favor of full-size tests. Profs. G. Lanza, R. H. Thurston, and William H. Burr have contributed valuable treatises on the subject of strength of timber. The extensive series of small and full size United States Government tests, conducted in 1880 to 1882 at the Watertown Arsenal, under Col. T. T. S. Laidley, and more recently the very elaborate and thorough timber tests being conducted by the United States Forestry Division under Dr. B. E. Fernow, chief, and Prof. J. B. Johnson, of Washington

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*Mr. Berg also kindly supplied in time for insertion in this publication the above report on "Strength of bridge and trestle timbers," to be read before the convention at New Orleans on October 16, 1895. As this comes in the shape of a recommendation from an international body regarding the future practice, it was considered desirable to make it a part of this bulletin.—B. E. F.

University, St. Louis, afford us to-day, in connection with the work of the above-mentioned experimenters, our most reliable data from a practical standpoint.

The test data at hand and the summary of criticisms of leading authorities seem to indicate the general correctness of the following conclusions:

(1) Of all structural materials used for bridges and trestles timber is the most variable as to the properties and strength of the different pieces classed as belonging to the same species; hence it is impossible to establish close and reliable limits for each species.

(2) The various names applied to one and the same species in different parts of the country lead to great confusion in classifying or applying results of tests.

(3) Variations in strength are generally directly proportional to the density or weight of timber.

(4) As a rule, a reduction of moisture is accompanied by an increase in strength; in other words, seasoned lumber is stronger than green lumber.

(5) Structures should be, in general, designed for the strength of green or moderately seasoned lumber of average quality and not for a high grade of well-seasoned material.

(6) Age and use do not destroy the strength of timber unless decay or season checking takes place.

(7) Timber, unlike materials of a more homogeneous nature, as iron and steel, has no well-defined limit of elasticity. As a rule, it can be strained very near to the breaking point without serious injury, which accounts for the continuous use of many timber structures with the material strained far beyond the usually accepted safe limits. On the other hand, sudden and frequently inexplicable failures of individual sticks at very low limits are liable to occur.

(8) Knots, even when sound and tight, are one of the most objectionable features of timber, both for beams and struts. The full-size tests of every experimenter have demonstrated not only that beams break at knots, but that invariably timber struts will fail at a knot or owing to the proximity of a knot, by reducing the effective area of the stick and causing curly and cross-grained fibers, thus exploding the old practical view that sound and tight knots are not detrimental to timber in compression.

(9) Excepting in top logs of a tree or very small and young timber, the heart wood is, as a rule, not as strong as the material farther away from the heart. This becomes more generally apparent, in practice, in large sticks with considerable heart wood cut from old trees in which the heart has begun to decay or been wind shaken. Beams cut from such material frequently season check along middle of beam and fail by longitudinal shearing.

(10) Top logs are not as strong as butt logs, provided the latter have sound timber.

(11) The results of compression tests are more uniform and vary less for one species of timber than any other kind of test; hence, if only one kind of test can be made, it would seem that a compressive test will furnish the most reliable comparative results.

(12) Long timber columns generally fail by lateral deflection or "buckling" when the length exceeds the least cross-sectional dimension of the stick by 20; in other words, when the column is longer than 20 diameters. In practice the unit stress for all columns over 15 diameters should be reduced in accordance with the various rules and formulæ established for long columns.

(13) Uneven end bearings and eccentric loading of columns produce more serious disturbances than are usually assumed.

(14) The tests of full-size long compound columns, composed of several sticks bolted and fastened together at intervals, show essentially the same ultimate unit resistance for the compound column as each component stick would have if considered as a column by itself.

(15) More attention should be given in practice to the proper proportioning of bearing areas; in other words, the compressive bearing resistance of timber with and

across grain, especially the latter, owing to the tendency of an excessive crushing stress across grain to indent the timber, thereby destroying the fiber and increasing the liability to speedy decay, especially when exposed to the weather and the continual working produced by moving loads.

The aim of your committee has been to examine the conflicting test data at hand, attributing the proper degree of importance to the various results and recommendations, and then to establish a set of units that can be accepted as fair average values, as far as known to-day, for the ordinary quality of each species of timber and corresponding to the usual conditions and sizes of timbers encountered in practice. The difficulties of executing such a task successfully can not be overrated, owing to the meagerness and frequently the indefiniteness of the available test data, and especially the great range of physical properties in different sticks of the same general species, not only due to the locality where it is grown, but also to the condition of the timber as regards the percentage of moisture, degree of seasoning, physical characteristics, grain, texture, proportion of hard and soft fibers, presence of knots, etc., all of which affect the question of strength.

Your committee recommends, upon the basis of the test data at hand at the present time, the average units for the ultimate breaking stresses of the principal timbers used in bridge and trestle constructions shown in the accompanying table.

In addition to the units given in the table, attention should be called to the latest formulæ for long timber columns, mentioned more particularly in the appendix to this report, which formulæ are based upon the results of the more recent full-size timber column tests, and hence should be considered more valuable than the older formulæ derived from a limited number of small-size tests. These new formulæ are Professor Burr's, Appendix I; Professor Ely's, Appendix J; Professor Stanwood's, Appendix K, and A. L. Johnson's, Appendix V; while C. Shaler Smith's formulæ will be better understood after examining the explanatory notes contained in Appendix L.

Attention should also be called to the necessity of examining the resistance of a beam to longitudinal shearing along the neutral axis, as beams under transverse loading frequently fail by longitudinal shearing in the place of transverse rupture.

In addition to the ultimate breaking unit stress the designer of a timber structure has to establish the safe allowable unit stress for the species of timber to be used. This will vary for each particular class of structures and individual conditions. The selection of the proper "factor of safety" is largely a question of personal judgment and experience, and offers the best opportunity for the display of analytical and practical ability on the part of the designer. It is difficult to give specific rules. The following are some of the controlling questions to be considered:

The class of structure, whether temporary or permanent, and the nature of the loading, whether dead or live. If live, then whether the application of the load is accompanied by severe dynamic shocks and pounding of the structure. Whether the assumed loading for calculations is the absolute maximum, rarely to be applied in practice, or a possibility that may frequently take place. Prolonged heavy, steady loading and also alternate tensile and compressive stresses in the same place will call for lower averages. Information as to whether the assumed breaking stresses are based on full-size or small-size tests or only on interpolated values, averaged from tests of similar species of timber, is valuable in order to attribute the proper degree of importance to recommended average values. The class of timber to be used and its condition and quality. Finally, the particular kind of strain the stick is to be subjected to and its position in the structure with regard to its importance and the possible damage that might be caused by its failure.

In order to present something definite on this subject, your committee presents the accompanying table, showing the average safe allowable working unit stresses for the principal bridge and trestle timbers, prepared to meet the average conditions existing in railroad timber structures, the units being based upon the ultimate

breaking unit stresses recommended by your committee and the following factors of safety, viz:

Tension with and across grain	10
Compression with grain	5
Compression across grain	4
Transverse rupture, extreme fiber stress	6
Transverse rupture, modulus of elasticity	2
Shearing with and cross grain	4

In conclusion, your committee desires to emphasize the importance and great value to the railroad companies of the country of the experimental work on the strength of American timbers being conducted by the Forestry Division of the United States Department of Agriculture, and to suggest that the American Association of Railway Superintendents of Bridges and Buildings indorse this view by official action and lend its aid in every way possible to encourage the vigorous continuance of this series of Government tests, which bids fair to become the most reliable and useful work on the subject of strength of American timbers ever undertaken. With additional and reliable information on this subject far-reaching economies in the designing of timber structures can be introduced, resulting not only in a great pecuniary saving to the railroad companies, but also offering a partial check to the enormous consumption of timber and the gradual diminution of our structural timber supply.

WALTER G. BERG, *Chairman.*
J. H. CUMMIN.
JOHN FOREMAN.
H. L. FRY.

Average safe allowable working unit stresses in pounds per square inch recommended by the committee on "strength of bridge and trestle timbers," American Association of Railway Superintendents of Bridges and Buildings, fifth annual convention, New Orleans, October, 1895.

Kind of timber.	Tension.		Compression.			Transverse rupture.		Shearing.	
			With grain.						
	With grain.	Across grain.	End bearing.	Columns under 15 diameters.	Across grain.	Extreme fiber stress.	Modulus of elasticity.	With grain.	Across grain.
Factor of safety	10	10	5	5	4	6	2	4	4
White oak	1,000	200	1,400	900	500	1,000	550,000	200	1,000
White pine	700	50	1,100	700	200	700	500,000	100	500
Southern, longleaf, or Georgia yellow pine	1,200	60	1,600	1,000	350	1,200	850,000	150	1,250
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	1,200	......	1,600	1,200	300	1,100	700,000	150	......
Red fir	1,000	......		......	......	800	......	......	......
Northern or shortleaf yellow pine	900	50	1,200	800	250	1,000	600,000	100	1,000
Red pine	900	50	1,200	800	200	800	600,000	......	......
Norway pine	800	......	1,200	800	200	700	600,000	......	......
Canadian (Ottawa) white pine	1,000	......	......	1,000	......	......	......	100	......
Canadian (Ontario) red pine	1,000	......	......	1,000	......	800	700,000	100	......
Spruce and Eastern fir	800	50	1,200	800	200	700	600,000	100	750
Hemlock	600	......	......	800	150	600	450,000	100	600
Cypress	600	......	1,200	800	200	800	450,000	......	......
Cedar	800	......	1,200	800	200	800	350,000	......	400
Chestnut	900	......	......	1,000	250	800	500,000	150	400
California redwood	700	......	......	800	200	750	350,000	100	......
California spruce	......	......	......	800	......	800	600,000	......	......

Average ultimate breaking unit stresses in pounds per square inch recommended by the committee on "strength of bridge and trestle timbers," American Association of Railway Superintendents of Bridges and Buildings, fifth annual convention, New Orleans, October, 1895.

Kind of timber.	Tension.		Compression.			Transverse rupture.		Shearing.	
			With grain.						
	With grain.	Across grain.	End bearing.	Columns under 15 diameters.	Across grain.	Extreme fiber stress.	Modulus of elasticity.	With grain.	Across grain.
White oak	10,000	2,000	7,000	4,500	2,000	6,000	1,100,000	800	4,000
White pine	7,000	500	5,500	3,500	800	4,000	1,100,000	400	2,000
Southern, longleaf, or Georgia yellow pine	12,000	600	8,000	5,000	1,400	7,000	1,700,000	600	5,000
Douglas, Oregon, and Washington fir or pine:									
Yellow fir	12,000	....	8,000	6,000	1,200	6,500	1,400,000	600	....
Red fir	10,000	....	....	....	....	5,000	....	....	....
Northern or shortleaf yellow pine	9,000	500	6,000	4,000	1,000	6,000	1,200,000	400	4,000
Red pine	9,000	500	6,000	4,000	800	5,000	1,200,000	....	....
Norway pine	8,000	....	6,000	4,000	800	4,000	1,200,000	....	....
Canadian (Ottawa) white pine	10,000	....	....	5,000	....	....	....	350	....
Canadian (Ontario) red pine	10,000	....	....	5,000	....	5,000	1,400,000	400	....
Spruce and Eastern fir	8,000	500	6,000	4,000	700	4,000	1,200,000	400	3,000
Hemlock	6,000	....	....	4,000	600	3,500	900,000	350	2,500
Cypress	6,000	....	6,000	4,000	700	5,000	900,000	....	....
Cedar	8,000	....	6,000	4,000	700	5,000	700,000	....	1,500
Chestnut	9,000	....	....	5,000	900	5,000	1,000,000	600	1,500
California redwood	7,000	....	....	4,000	800	4,500	700,000	400	....
California spruce	....	....	....	4,000	....	5,000	1,200,000	....	....

NOTE.—These and the following tables are printed here as part of the preceding report. In doing so the Forestry Division disclaims any apparent indorsement of the data contained therein, except as far as its own results are recorded; the other data having been probably obtained from a small number of tests without reference to and allowance for the conditions of the material as to state of seasoning, etc., believed to be an essential requisite.
B. E. F.

STRENGTH OF BRIDGE

TABLE I.—

[Ultimate breaking stress

RECOMMENDED

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
W. J. M. Rankine	A	Red oak		10,250				6,000
		English oak	10,000–19,800					10,000
Chas. H. Haswell	A	Oak				2,300		
		White oak		16,500				7,500
		Live oak		16,380				6,850
		Canadian white oak.						5,982
		Red oak		10,250				
		Pennsylvania oak, seasoned.		20,333				
John C. Trautwine	A	Oak				2,300	5,000–7,000	6,000
		White oak		10,000				
		Live oak		10,000				
		Basket, black, and red oak.		10,000				
Robert H. Thurston	A	Oak						
		White oak		10,000			5,500–8,000	
		Live oak		10,000			8,000–10,000	
		Canadian oak						
Louis De C. Berg	A	White oak		11,000		2,300		7,200
		Red oak		8,000				6,000
		Live oak		11,000				6,850
		Canadian oak		7,500				6,000
F. E. Kidder	A	White oak		16,000			3,150–7,000	
Malverd A. Howe	A	White oak		10,000				7,000
		Live oak		10,000				7,000
William Kent	A	Oak						
A. L. Johnson	W	White oak		10,000				4,000

*Compiled for the Fifth Annual Convention of the American Association of Railway

RESULTS OF

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
U. S. Ordnance Department, Capt. T. J. Rodman.	B i	White oak, well seasoned.	13,333–25,222				4,691–10,058	
Thomas Laslett	B b	White oak		7,021				6,964
	B b	Baltimore oak		3,832				5,891
R. G. Hatfield	B a	Oaks, average						
		White oak		19,500			6,531–9,775	8,000
		Canadian oak						
		Live oak						11,100
U. S. Tenth Census	B h	White oak					5,810–9,070	
		White, post, iron, red, and black oak.						7,000
		Scrub and basket oak.						6,000
		Chestnut and live oak.						7,500
		Pin oak						6,500
Robert H. Thurston	B c	White oak		13,210				7,140
		Live oak		10,310				10,410
St. Louis Bridge	B d	White oak:						
		Blocks					3,200–3,778	3,505
		Round columns					6,000–12,200	7,812
		Black oak:						
		Blocks						
		Round columns					5,400–6,980	6,101
U. S. Ordnance Department, Watertown Arsenal.	B f	White oak	12,670–22,703	17,410				7,192
		Red oak	7,600–12,133	10,124				
		Yellow oak	20,260–20,520	20,390				

AND TRESTLE TIMBERS.*

Oak.

in pounds per square inch.]

VALUES.

Compression. Across grain. Limits.	Compression. Across grain. Indentation.	Compression. Across grain. Average.	Transverse rupture. Extreme fiber stress. Limits.	Transverse rupture. Extreme fiber stress. Average.	Transverse rupture. Modulus of elasticity. Limits.	Transverse rupture. Modulus of elasticity. Average.	Shearing. With grain. Limits.	Shearing. With grain. Average.	Shearing. Across grain. Limits.	Shearing. Across grain. Average.
				10,600						
			10,000–13,600		1,200,000–1,750,000			2,300		4,000
		2,300				1,710,000		780		4,032
				10,800						
				11,520						
				10,512						
				9,720						
					1,000,000–2,000,000	1,500,000	400–700			
				10,800						4,425
				10,800						8,480
				15,300						
								780		4,000
				11,000						
				12,000						
				10,000						
		2,400		7,200		900,000		800		4,400
				6,500		1,200,000		750		
		4,500		7,300				700		8,500
				7,000						
				5,670		1,240,000		780		4,400
	1/10"	1,600				1,500,000		850		4,400
	1/10"	1,600				1,500,000				
					974,000–2,283,000					
	3/8	1,200		6,000		1,100,000		800		

Superintendents of Bridges and Buildings, October, 1895, by Walter G. Berg.

SMALL-SIZE TESTS.

Compression. Across grain. Limits.	Compression. Across grain. Indentation.	Compression. Across grain. Average.	Transverse rupture. Extreme fiber stress. Limits.	Transverse rupture. Extreme fiber stress. Average.	Transverse rupture. Modulus of elasticity. Limits.	Transverse rupture. Modulus of elasticity. Average.	Shearing. With grain. Limits.	Shearing. With grain. Average.	Shearing. Across grain. Limits.	Shearing. Across grain. Average.
			8,460–17,340							
				10,900		1,330,000				
				9,800		1,770,000				
				8,550		1,114,560				
	1/20"	2,650		11,700		1,339,200	1,076–1,474	1,250		
				10,600		1,929,312				
	1/20"	6,800								
	{ 1/100"	1,600 }	7,010–18,360		879,000–2,103,000					
	1/10"	4,000 }								
	1/10"	4,000								
	1/10"	4,200								
	1/10"	4,500								
	1/10"	3,000								
				9,840		1,620,000				
				11,280		1,851,428				
1,300–2,200		1,750								
1,600–2,000		1,800								
	1/20"	2,850						842		
								803		

TABLE I.—

[Ultimate breaking stress

RESULTS OF

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
G. Lanza	......	White oak:						
	D c	36 beams	....	......	......	......	....	......
	G a	10 posts and blocks.	....	......	......	......	3,132–4,450	3,470
	G a	18 old posts	....	......	......	......	2,943–6,147	3,957
D. Kirkaldy & Son	E b	White oak:						
		5 beams, 5 feet span.	....	......	......	......	....	......
		5 beams, 11 feet span.	....	......	......	......	....	......
		5 posts, about 6 diameters.	....	......	......	......	....	3,285
		5 posts, about 11 diameters.	....	......	......	......	....	3,418
		5 posts, about 23 diameters.	....	......	......	......	....	2,891

Oak—Continued.

in pounds per square inch.]

FULL-SIZE TESTS.

Compression.			Transverse rupture.				Shearing.			
Across grain.			Extreme fiber stress.		Modulus of elasticity.		With grain.		Across grain.	
Limits.	Indentation.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.
....	......	......	3,535-7,834	5,863	744,774-1,777,500	1,131,100	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......			....	......	....	......	......	......
....	......	......	....	6,890	....	......	....	......	......	......
....	......	......	....	8,550	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......

TABLE II.—

[Ultimate breaking stress

RECOMMENDED

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
Chas. H. Haswell	A	White pine		11,800		550		5,775
John C. Trautwine	A	White pine		10,000		550	5,000–7,000	6,000
Robert H. Thurston	A	White pine	3,000–7,500				3,000–6,000	
Louis De C. Berg	A	White pine		9,000		550		5,500
F. E. Kidder	A	White pine		7,000			2,800–4,500	
Malverd A. Howe	A	White pine		10,000				5,400
H. T. Bovey	M	Canadian (Ottawa) white pine.						5,000
A. L. Johnson	W	White pine		7,000				3,500
W. M. Patton	A	White pine		7,000				9,500

RESULTS OF

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
U. S. Ordnance Department, Capt. T. J. Rodman.	B i	White pine, well seasoned.	11,433–11,960				5,017–5,775	
R. G. Hatfield	B a	White pine		12,000			5,579–7,502	6,650
U. S. Tenth Census	B h	White pine					3,750–5,600	5,400
Robert H. Thurston	B c	White pine		6,880				9,590
St. Louis Bridge	B d	White pine:						
		Blocks					3,083–3,694	3,261
		Columns					3,580–3,900	3,727
F. E. Kidder	B e	White pine						
U. S. Ordnance Department, Watertown Arsenal.	B f	White pine	5,300–11,299	8,916				5,617
	Q	White pine, resistance to keys tearing out.						
H. T. Bovey	M	Canadian (Ottawa) white pine.	8,503–14,273	11,396				

RESULTS OF

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
H. T. Bovey		Canadian (Ottawa) white pine.						
	M i	15 beams						
	M ii	68 posts						3,843
U. S. Ordnance Department, Watertown Arsenal.	H a	White pine:						
		Posts under 32 diameters.					1,687–3,700	2,414
		Posts 32–62 diameters.					1,000–2,000	
G. Lanza	D d	White pine, 37 beams.						
		Western white pine, kiln dried, 8 beams.						
Onward Bates		White pine:						
	R a	New and old, 30 beams.						
	R b	New, 14 beams						
W. H. Finley	S	White pine:						
		31 years in use, 12 beams.						
		New, 2 beams						

White pine.

in pounds per square inch.]

Compression.			Transverse rupture.				Shearing.			
Across grain.			Extreme fiber stress.		Modulus of elasticity.		With grain.		Across grain.	
Limits.	Indentation.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.
VALUES.										
		550		9,000		1,830,000		490		
				8,100		1,600,000	250–500			2,480
						1,000,000		490		
		700		4,000		850,000		450		2,500
				4,320		1,073,000		490		2,480
	1/10"	600				1,750,000		325		2,480
	3%	440		4,400		870,000		300		
				9,000				482		2,480
SMALL-SIZE TESTS.										
			6,798–7,092							
	1/20"	800		9,000		1,252,800	433–530	480		
	1/100"	600	5,610–11,530		868,000–1,478,000					
	1/10"	1,200								
				5,280		883,636				
555–722		611								
			7,578–9,440	8,297	1,251,252–1,461,728	1,388,497				
	1/20"	1,045						370		
							236–611	421		
							273–382			
FULL-SIZE TESTS.										
			2,500–4,936	3,388	433,250–1,184,240	754–265				
			2,456–7,251	4,451	727,200–1,565,000	1,222,000				
				5,482		1,183,037				
			2,350–5,376	3,872	712,500–1,430,900	1,098,000				
			2,160–5,131	3,694						
			5,139–10,616	7,051	715,000–1,900,000	1,208,250				
				5,402		982,500				

Table III.—*Southern yellow pine, longleaf yellow*

[Ultimate breaking stress

RECOMMENDED

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
W. J. M. Rankine	A	Yellow pine						5,400
Chas. H. Haswell	A	Pitch pine				550		8,947
		Yellow pine		13,000		550		
		Virginia pine		19,200		550		8,200
		Georgia pine				550		
John C. Trautwine	A	Yellow pine		10,000		550		
		Georgia yellow pine.						
		Pitch pine		10,000				
Robert H. Thurston	A	Yellow pine	5,000–12,000				6,500–10,000	
		Pitch pine	8,000–10,000					
Louis De C. Berg	A	Yellow pine		9,000				5,400
		Georgia yellow pine.		12,000				7,400
		Pitch pine		10,000				8,900
F. E. Kidder	A	Yellow pine		16,000			4,400–6,000	
Malverd A. Howe	A	Southern yellow pine.		10,000				8,500
G. Lanza	D b	Yellow pine						
A. L. Johnson	W	Longleaf pine		12,000				5,000
W. M. Patton	A	Yellow pine		20,700				11,500
RESULTS OF								
U. S. Ordnance Department, Capt. T. J. Rodman.	B i	Yellow pine, well seasoned.	12,600–19,200				7,836–8,350	
Thomas Laslett	B b	Pitch pine		4,666				6,462
R. G. Hatfield	B a	Georgia pine		16,000			8,170–11,503	9,500
		Pitch pine						
U. S. Tenth Census	B h	Longleaf Georgia pine.					4,010–10,600	8,500
Robert H. Thurston	B c	Yellow pine		20,700				11,950
St. Louis Bridge	......	Yellow pine:						
	B d	Blocks					4,500–4,917	4,722
		Columns					4,650–4,820	4,735
F. E. Kidder	B e	Yellow pine						
U. S. Ordnance Department, Watertown Arsenal.	B f	Yellow pine	12,066–17,922	15,478				
	Q	Yellow pine, resistance to keys tearing out.						
U. S. Forestry Division, Bulletin No. 8.	C	Longleaf pine, from Alabama.	4,170–31,890	16,029			4,587–9,850	7,228
RESULTS OF								
U. S. Forestry Division, Bulletin No. 8.	C	Longleaf pine, from Alabama.						
G. Lanza	......	Yellow pine:						
	D b	51 beams						
	G a	18 posts and blocks.					3,604–5,452	4,544
U. S. Ordnance Department, Watertown Arsenal.	......	Yellow pine:						
	H a	Posts under 22 diameters.					3,430–5,677	4,442
	H a	Posts 22 to 62 diameters.					1,700–3,500	
	H c	Straight grained, well seasoned, 12 posts.					5,593–8,644	7,386
		Very slow growth, 3 posts.					7,820–10,250	9,339
		Very green and wet, 3 posts.					2,795–3,180	3,015

pine, Georgia yellow pine, or Southern pitch pine.

in pounds per square inch.]

VALUES.

Compression.			Transverse rupture.				Shearing.			
Across grain.			Extreme fiber stress.		Modulus of elasticity.		With grain.		Across grain.	
Limits.	Indentation.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.
....	......	......	....	......	....	......	....	......	......	......
....	......	550	....	9,864	....	2,430,000	....	510	......	......
....	......	550	....	9,360	....	......	....	......	......	......
....	......	550	....	......	....	......	....	......	......	......
....	......	550	....	14,400	....	......	....	......	......	......
....	......	......	....	9,000	....	1,600,000	....	......	4,340–5,735	......
....	......	......	....	15,300	....	......	....	......	......	5,735
....	......	......	....	9,900	....	......	....	......	......	5,053
....	......	......	....	7,000	....	1,600,000	....	510	......	......
....	......	......	....	8,000	....	1,900,000	....	......	......	......
....	......	......	....	6,000	....	1,100,000	....	......	......	4,800
....	......	1,850	....	7,200	....	1,200,000	....	500	......	5,700
....	......	......	....	6,600	....	1,225,000	....	510	......	5,000
....	......	......	....	6,750	....	1,780,000	....	510	......	5,700
....	1/10"	1,300	....	......	....	1,600,000	....	325	......	5,700
....	......	......	....	5,000	....	......	....	......	......	......
....	3%	645	....	7,750	....	1,440,000	....	500	......	......
....	......	......	....	15,000	....	......	....	843	......	5,735
SMALL-SIZE TESTS.										
....	......	......	8,796–11,676	9,972	....	......	....	......	......	......
....	......	......	....	11,900	....	1,900,000	....	......	......	......
....	1/20"	2,250	9,000–21,168	15,300	....	2,468,800	713–934	840	......	......
....	......	......	....	9,792	....	1,225,152	....	......	......	......
....	1/100" 1/10"	1,300 2,600	9,220–21,060	......	879,000–2,878,000	......	....	......	......	......
....	......	......	....	16,740	....	3,534,727	....	......	......	......
1,000–1,222	......	1,092	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	12,280–14,654	13,048	1,707,282–1,926,160	1,821,630	....	......	......	......
....	1/20"	1,900	....	......	....	......	....	352	......	......
....	......	......	....	......	....	......	337–720	512	......	......
584–2,094	15%	1,517	....	......	....	......	464–1,299	852	......	......
FULL-SIZE TESTS.										
....	......	......	4,268–16,200	12,250	842,000–3,117,370	2,069,650	....	......	......	......
....	......	......	3,963–11,360	7,486	1,162,467–2,386,096	1,757,900	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......
....	......	......	....	......	....	......	....	......	......	......

TABLE IV.—*Douglas, Oregon, Washington,*

[Ultimate breaking stress

RECOMMENDED

Authority.	Appendix reference.	Description.	Tension. With grain. Limits.	Tension. With grain. Average.	Tension. Across grain. Limits.	Tension. Across grain. Average.	Compression. With grain. Limits.	Compression. With grain. Average.
Robert H. Thurston	A	Oregon pine	9,000-14,000				9,200-11,500	
		California spruce.	12,000-14,000				9,200-12,800	
F. E. Kidder	A	Oregon pine		13,800				
		Oregon spruce						
H. T. Bovey	M	Douglas fir						6,000
		Douglas fir, specially selected.						
		Douglas fir, ordinary first quality.						
Arthur Brown, Southern Pacific Ry.	O b	Pacific Northwest (Douglas) fir.		15,900				6,000
A. L. Johnson	W	Douglas fir						4,400
RESULTS OF								
U. S. Ordnance Department, Capt. T. J. Rodman.	B i	Oregon yellow fir.	13,633-16,833				7,488-9,217	
		Oregon red fir		12,867				7,083
		Oregon white fir		14,533				6,644
U. S. Tenth Census	B h	Red and yellow Douglas fir.					4,880-9,800	
W. B. Wright	O d	Red fir		10,872				6,099
		Yellow fir		11,550				6,132
Oregon and California R. R.	O e	Douglas fir		16,600				3,085
		Oregonsugarpine.		11,000				3,391
Robert H. Thurston	O c	Oregon pine					9,200-11,500	
		California spruce.					9,200-12,800	
U. S. Ordnance Department, Watertown Arsenal.	B f	Oregon pine		13,810				8,597
		Oregon spruce		16,160				5,772
H. T. Bovey	M III	Douglas fir, 71 tests.	2,485-18,856	11,612				
	M IV	Douglas fir						
John D. Isaacs, Southern Pacific Ry.	T	Douglas fir		15,900				6,000
RESULTS OF								
Onward Bates		Douglas fir:						
	B i	12 beams						
		10 beams (omitting 2 bad sticks).						
H. T. Bovey		Douglas fir:						
	M I	Specially selected, 4 beams.						
	M I	Ordinary first quality, 15 beams.						
	M II	122 posts						5,974
		British Columbia spruce:						
	M I	3 beams						
	M II	69 posts						3,617
C. B. Talbot, Northern Pacific R. R.	N a	Washington yellow fir, 5 small beams.						
	N e	Washington yellow fir, hard fine grain, 1 small beam.						
	N e	Washingtonpine, fine grain, 1 small beam.						

and California fir, pine, and spruce.

in pounds per square inch.]

VALUES.

Compression.			Transverse rupture.				Shearing.			
Across grain.			Extreme fiber stress.		Modulus of elasticity.		With grain.		Across grain.	
Limits.	Indentation.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.
				11,071						
				12,228						
								840		
								310		
								400		
				9,000		2,000,000				
				6,000		1,430,000				
				13,630		1,272,000		600		
	3%	500		6,600		1,380,000				

SMALL-SIZE TESTS.

Compression across grain: Limits.	Indentation.	Average.	Extreme fiber stress: Limits.	Average.	Modulus of elasticity: Limits.	Average.	Shearing with grain: Limits.	Average.	Shearing across grain: Limits.	Average.
			7,740–10,944							
				6,728						
				4,194						
			8,220–17,920		1,308,000–2,579,000					
				15,894						
				15,030						
	$\frac{3}{100}$″	1,000		8,658			515–833	689		
	$\frac{4}{100}$″	1,000		8,370						
				11,071						
				12,228						
	$\frac{1}{20}$″	1,150		17,223				786		
	$\frac{1}{20}$″	695						311		
							377–411	403		
						1,272,000		600		

FULL-SIZE TESTS.

Compression across grain: Limits.	Indentation.	Average.	Extreme fiber stress: Limits.	Average.	Modulus of elasticity: Limits.	Average.	Shearing with grain: Limits.	Average.	Shearing across grain: Limits.	Average.
			3,597–7,544	5,791						
			5,268–7,544	6,214						
			8,020–10,441	9,054	1,934,500–2,178,100	2,036,529				
			4,027–8,382	6,081	926,500–1,770,563	1,431,209				
			4,614–5,908	5,120	1,011,450–1,528,499	1,203,633				
			6,890–9,720	7,847						
				9,720						
				5,116						

TABLE IV.—*Douglas, Oregon, Washington, and*

[Ultimate breaking stress

RESULTS OF FULL-

Authority.	Appendix reference.	Description.	Tension.				Compression.	
			With grain.		Across grain.		With grain.	
			Limits.	Average.	Limits.	Average.	Limits.	Average.
A. J. Hart, Chic., Milw. and St. Paul R. R.	N b	Washington yellow fir:						
		green, 4 beams.						
		6 years seasoned, 2 beams.						
A. J. Hart and C. B. Talbot.	N c	Washington fir, 9 beams.						
S. Kedzie Smith	N d	Washington yellow fir, close grain, 2 beams.						
	N d	Washington red fir, 8 beams.						
	O a	Washington yellow and red fir, 19 beams.						
Report Washington Chapter Amer. Inst. Architects.	N f	Douglas fir, 11 beams.						
		Washington yellow fir, 13 beams.						
		Washington red fir, 11 beams.						
		Average of all tests.						
Charles B. Wing	U	Douglas fir, ordinary No. 1 merchantable:						
		10 beams						
		10 small beams.						

California fir, pine, and spruce—Continued.

in pounds per square inch.]

SIZE TESTS—Continued.

Compression.			Transverse rupture.				Shearing.			
Across grain.			Extreme fiber stress.		Modulus of elasticity.		With grain.		Across grain.	
Limits.	Indentation.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.	Limits.	Average.
......	......	......	6,143–7,982	7,323	......	......	......	......	......	......
......	......	......	5,935–6,088	6,020	......	......	......	......	......	......
......	......	......	5,263–7,561	6,273	......	......	......	......	......	......
......	......	......	7,500–8,160	7,830	......	......	......	......	......	......
......	......	......	4,255–6,138	5,186	......	......	......	......	......	......
......	......	......	3,530–8,160	5,420	......	......	......	......	......	......
......	......	......	......	5,979	......	......	......	......	......	......
......	......	......	......	7,402	......	......	......	......	......	......
......	......	......	......	5,186	......	......	......	......	......	......
......	......	......	......	6,359	......	......	......	......	......	......
......	......	......	5,580–7,951	6,482	......	......	......	......	......	......
......	......	......	6,138–12,056	9,257	......	......	......	......	......	......